RIVER AND CANAL ENGINEERING

THE CHARACTERISTICS OF OPEN FLOWING STREAMS, AND THE PRINCIPLES AND METHODS TO BE FOLLOWED IN DEALING WITH THEM

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PREFACE

The scope of this book is described on the first page. Though called a second edition it is really a new book. The first edition — hurriedly prepared and long since exhausted — contained only about a third of the matter of the present volume.

The book embodies the experience of a great number of years spent among streams. It is hoped that it will be of use to the student of general engineering and to the civil engineer who is concerned with rivers and streams, canals or reservoirs.

There are of course many items which have to be mentioned in more than one chapter or article. When it is desired to look up any such item completely, reference should be made to the Index.

The references to *Hydraulics* are to the author's *Hydraulics with Working Tables*. In it special attention is given to the hydraulics of open streams.

GUILDFORD, 1st JULY 1924.

E. S. B.

CHAPTER I

INTRODUCTION

Art. 1. Preliminary. River and Canal Engineering is that branch of engineering science which deals with the characteristics and tendencies of streams flowing in open channels, and with the principles and methods which should be followed in works for dealing with, altering and controlling them or for storing or utilising their waters.

The ultimate object of a work may be the development or improvement of navigation, hydro-electric power, drainage, or irrigation; or the mitigation of floods or of other injurious action of a stream. Each division of the present treatise — as indicated by the headings of the chapters or articles — generally treats of works of a particular class rather than of those having one ultimate object. Reservoirs for instance — or artificial channels or weirs — may be constructed with various objects but the principles and general methods of construction are the same for all and it is obviously desirable to deal with them once for all, any special points depending on the ultimate object of the work being, however, duly attended to.

A work may, of course, be of any degree of magnitude, but the same principles apply to all. A work of one class is often combined with one of another.

Before any considerable work in connection with a stream can be undertaken — and often before it can even be decided whether or not it is to be undertaken — it is necessary to carefully study the stream, perhaps for a long period, and to collect information concerning it. The information which is required depends on the character of the stream and on the nature of the work which is to be done.

In very many cases — particularly in connection with drainage, floods or storage — the question of rainfall has to be studied, or that of the quantity and nature of the solids transported by the stream. A few general items as to which information and study are very frequently required, will be considered directly (Art. 2). Works when constructed have to be maintained. The study of the stream and the collection of information concerning it, have to go on even when it is only a question of maintenance.

After obtaining information concerning the stream to be dealt with, careful calculations are, in the case of any large and important work, made as to the effects which will be produced by it. In many cases these effects cannot be exactly foreseen. Sometimes matters can be arranged so that the work can be stopped short at some stage without destroying the utility of the portion done, or so that the completed work can be altered to some extent. An embankment, for instance, can be raised or extended.

In most works for controlling streams there is, as will appear in due course, a considerable choice of types of work and methods of construction. In practice it is often necessary to adopt one particular type or kind of work because only certain kinds of materials can be obtained cheaply and readily on the spot. Similar considerations apply to repairs. In all works — whether of construction or maintenance — where water is concerned, care and vigilance are required.

It has sometimes been said that perishable materials, such as trees, stakes, and brushwood, when used in works, cannot produce permanent results. Such statements are largely erroneous. By the time the materials have decayed, the changes wrought may have been very great, deposits of shingle or silt may have occurred and become covered with vegetation, and there may be little or no tendency for matters to revert to their former condition. The use of perishable materials may be justified even if renewals will probably be necessary. If the expense of using more lasting materials had had to be incurred, the works might never have been carried out at all. On the Mississippi enormous

quantities of work have been done with fascines. There are, however, cases in which the use of perishable materials is uneconomical. If, for instance, a spur is made of alternate layers of brushwood and stone, the decay of the brushwood may involve the loss of the stone. Wood and brushwood are liable to be quickly destroyed in salt water by the teredo.

Regarding the units employed by hydraulic engineers, complaints are often made as to their variability. It is highly desirable to adopt a uniform system. Discharges should be given in cubic feet per second and velocities in feet per second. The discharges from catchment areas should be given in cubic feet per second per square mile. The square mile is generally used in the United States and very often in England, but as a unit of 1,000 acres is often used, and is in some cases convenient, a table showing the corresponding figures is given below.

The following approximations, much used by engineers, enable a saving of time to be effected in calculations and are often sufficiently accurate. The number of seconds in a day is 86,400. The number of square feet in an acre is 43,560 or very little more than half the above. Assuming it to be half, a discharge of 1 cubic foot per second for a day gives 2 acre-feet, that is, it will cover an acre of ground to a depth of 2 feet in a day; and in six months it will give 365 acre-feet, or will cover 100 acres to a depth of 3.65 feet. The acre-foot is frequently used as a unit.

Another method of using the figures is as follows. An acre contains 43,560 square feet and a twelfth of this is 3,630. Therefore a run-off of one-twelfth of a foot of rain in an hour, gives a discharge per acre of 3,630 cubic feet per hour. Or — approximately — 1 inch of run-off from 1 acre in 1 hour, gives one cubic foot per second. This is 640 cubic feet per second for a square mile.

To obtain almost complete accuracy a correction of 1 per cent can be made to the figure arrived at.

Square Miles.	Acres.
1	640
1.5625	1,000

Square Miles.	Acres.
2	1,280
3	1,920
3.125	2,000
4	2,560
4.6875	3,000
5	3,200
6	3,840
6.25	4,000
7	4,480
7.8125	5,000
8	5,120
9	5,760
9.375	6,000
10	6,400
10.9375	7,000
11	7,040
12	7,680
12.5	8,000
13	8,320
14	8,960
14.0625	9,000
15	9,600
15.625	10.000

Art. 2. Water Levels and Discharges. When any work on a river is to be carried out, it is nearly always necessary to know the approximate highest and lowest water-levels. These can often be ascertained by local enquiry, combined with observations of water marks, but in very many cases it is necessary to know how the water-level varies from day to day, so as to be able to construct a diagram showing the gauge readings as ordinates, the abscissæ being the times in days starting from any convenient date as zero. Such a diagram or "hydrograph" is given in Chap. VII., Art. 1. It is often convenient to calculate the average readings for periods of 5, 10 or 20 days and to show only these averages.

Making a survey and plan, and laying down on it the

lines for longitudinal and cross-sections, and taking levels for the sections, are ordinary operations of surveying. If any land is liable to be flooded, its boundaries should be shown on the plan. Unless the water is shallow, it is necessary to obtain the bed levels from the water-level by soundings, the level of a stake driven down to the water-level having been obtained by levelling. All the sections should show the water-level as it was at some particular time, but the water-level will probably have altered while the survey was in progress, and the allowance for this must be carefully made The stakes at all the cross-sections and on both banks of the stream — for the water-levels at opposite banks may not be exactly the same — may, for instance, be driven down to the water-level when it is steady, and thereafter any changes in it noted and the soundings corrected accordingly.

In order to ascertain what changes are occurring in the channel it may be necessary to repeat the soundings at intervals and, if there is much erosion of the bank, to observe it.

The methods and instruments to be employed for making observations of water-levels and discharges, and the most suitable sites for discharge observations, are fully considered in *Hydraulics* and it is there stated that if at high water, or during a flood, soundings cannot be taken, they must be inferred from those taken previously or afterwards — or both — at lower stages of the stream; and that if the velocities cannot be observed they must be calculated from surface slope observations Various values of N (for Kutter's co-efficients) are also given and full tables of the co-efficients It remains to consider the question of the systematic observation of water-levels and discharges at given sites and to deal with certain difficulties which may arise. For the present it is assumed that the stream is a large one In a case in which the local surface slope cannot be observed during a flood, it can be inferred or calculated approximately (*Hydraulics*, Chap VII. Art. 15, para. 3).

Unless the stream being dealt with is an artificial one,

it is unlikely that the flow in any reach with which the work is concerned, will be uniform. When, in a non-uniform stream, a rise of the surface occurs at a given point and the flow then becomes steady, there will be, in due course, a rise at a given point further down, but it is improbable that this rise, even when the flow there has become steady, will be the same as at the upstream point. The rise and fall of the water at one place cannot therefore be correctly inferred from those at another. It will be desirable to have two gauges, either read daily or else automatically recording the water-level, one near each end of the reach concerned, with intermediate gauges if the reach is very long. If, in or near the reach, there is already a gauge which has been regularly read, it may be sufficient to set up only one new gauge.

If the channel is quite stable, whether it is uniform or not, and there are no disturbing influences — referred to again below — at or downstream of a discharge site, a given gauge reading at that site corresponds to a given discharge, and as soon as the discharges corresponding to a few given water-levels have been ascertained it is possible, by plotting the discharges as ordinates, the gauge readings being the abscissæ, to draw a discharge curve and from it construct a discharge table. There are likely to be some discrepancies among the observed discharges, so that a regular curve will not pass through all the plotted points. It is made to pass as near to them as possible.

But in the case of a stream whose channel is liable to alter by silting or scour, the discharge probably alters while the water-level remains steady. When this condition is suspected to exist, the discharge should be observed frequently If it is found that the discharge alters while the water-level and cross-section of stream are the same, the change is in the velocity and this will need investigation. It may be due to a change of surface slope, owing to silting or scour downstream of the site, or to a change in roughness. A deposit of muddy silt is likely to give a smooth channel. Scour may roughen a channel As to the effect of suspended silt on velocity

see Chap. III., Art. 2. It may be found that there is a tendency for one particular kind of change to affect a given site. The discharge may for instance steadily increase. In such a case there will be, not one but a series of discharge curves each appertaining to a particular date or period. If on the other hand the discharge — owing say to alternate silting and scour — fluctuates while maintaining a steady average, it may be possible to construct only one discharge curve but the discrepancies in the ordinates may be considerable and the figures more or less rough.

On the river Irrawaddy the Henzada gauge is some 73 miles downstream of the Saiktha gauge. A record flood occurred at Henzada. Much discussion occurred as to whether a railway embankment and a marginal embankment had raised the flood level. Opinions differed but it seems to have been assumed that a reading on the Saiktha gauge was a good indication of the flood discharge. The readings on that gauge had, in the course of 40 years fallen 2.11 feet with reference to corresponding readings at Henzada [1]. If no information exists except the readings on two such gauges, a question such as the above is indeterminate. In the case under discussion there were, no doubt, reasons for the opinions held, but decisive information in such cases can only be obtained from observations of the discharge of the channel.

A discharge site or gauge site should, if possible, be beyond the influence of any disturbing occurrences — at points downstream of the site — which cause the water to be temporarily headed up or drawn down. If it is within their influence the water-level for a given discharge is variable and the case, as regards the discharge curve, is similar to what occurs when changes take place in the channel. Such disturbing occurrences are the manipulation of sluice gates or the like, variations in the inflow of water from tributary streams or from flood water, or in the drawing off of water, or in the level of any body of water into which the stream debouches. The distance upstream to which any such in-

[1] *Note on the Irrawaddy River*, Samuelson Government Press, Rangoon.

fluence extends can now be calculated, if the case is one of heading up, without much difficulty in a fairly uniform stream. In a case of draw-down or in an irregular channel there is some difficulty[1]. The distances are sometimes known beforehand or can be observed. Cases of heading up are by far the more common of the two and generally the more important.

Other possible causes of disturbance are changes in the quantities of water lost or gained in a given reach (Chap. VI., Art. 2).

The preceding observations cover cases in which the stream is large and accuracy is required. In some cases it is suitable to disregard irregularities or changes in the channel and to treat the stream as if it were uniform and stable.

In cases where there are sluices — or other works with a considerable fall in the water surface — the discharge can be calculated if the head is observed and if the coefficients are known with sufficient accuracy. If the stream is small enough the discharges can be ascertained by means of a weir of planks. The discharge is then known from the gauge reading, which can as before be automatically recorded. In many cases, as will be seen, such weirs cannot be constructed and it is necessary to rely almost entirely on rainfall figures.

In a shifting river (Chap. III., Art. 5) a gauge has often to be moved because the site becomes silted up or because the bank is being eroded daily. In India a cross line is marked out at right angles to the general direction of the river and as far as possible the gauge is kept in that line, its zero being at a given level. When the gauge is shifted out of the line, the zero level is altered so that the new gauge, on the day it is shifted, reads the same as the old one. This is also done at any subsequent shifting. When the gauge is shifted back into the original cross line with its zero at the original level, there is usually a sudden change in the reading because of the changes in the surface slopes which

[1] *Hydraulics*, Chap. VII. Arts. 13 and 15.

have occurred since it was last in the line. In order to minimise the amount of such a change in the reading, the water-level in the fixed line is observed every year in October — that is after the floods — and the zero level of the gauge is adjusted so as to make it read the same as it would have read if in the line. In the absence of the above procedure the readings of the gauge taken over a long period of time, cannot be said to refer to any particular point on the course of the stream.

CHAPTER II

RAINFALL, EVAPORATION AND ABSORPTION

Art. 1 Rainfall. Rain is brought by winds which blow across the sea. Hence the rainfall in any country is generally greatest in those localities where the prevailing winds blow from seaward, provided they have travelled a great distance over the sea. Rainfall is greater among hills than elsewhere, because the temperature at great elevations is lower. Currents of moist air striking the hills are deflected upwards, become rarefied and cooled, and the water vapour condenses and becomes rain. This process, if the hill tract is not broad and lofty, may not produce its full effect till the air currents have passed over the hills, and thus the rainfall on the leeward slopes may be greater than elsewhere, but on the inner and more lofty ranges the rainfall is generally greatest on the windward side. In South Africa it has been found that mountain ranges have little effect on the rainfall, the reason being that most of the rain occurs in thunderstorms [1]. The rainfall at any place, in any year, is of course the sum of the daily falls. This annual fall varies so much, even at the same place, that the figure for any one year is of limited value. A figure very frequently required by the engineer is the mean annual rainfall, that is the average of the annual falls extending over very many years.

The mean annual rainfall varies very greatly according to the locality. In England it varies from about 20 inches at Hunstanton in Norfolk, to about 135 inches at Seathwaite in Cumberland; in India, from 2 or 3 inches in parts of Sind to 600 inches or more at Cherrapunji — 4,000 feet above the sea — in the Eastern Himalayas. It is in this neigh-

[1] *Proceedings, Royal Soc. of S. Africa*, Vol. 9.

bourhood that the monsoon winds, obstructed by the Khasia hills, are deflected suddenly to a height of 6,000 feet.

Places near the ocean generally have a higher rainfall than those which are remote from it or are separated from it by high mountain ranges.

As regards local topography, it is roughly correct to say that in any given locality the greater the elevation the greater the rainfall but there are many exceptions to this. Among hills the rainfall may vary greatly at places not far apart. An extreme instance of this occurs in the Bombay hills, where the mean annual falls at two stations only ten miles apart are respectively 300 inches and 50 inches. The difference is due to difference in elevation as well as to situation. The question of topography is further dealt with in Art. 5.

In temperate climates the rainfall is generally distributed over all the months of the year and a division of the year into wet and dry periods is not easy. In the west of the United States, however, there are such periods, five summer months on the Pacific coast being nearly rainless. In tropical and sub-tropical regions the great bulk of the rain often falls in two or three months and there are long dry periods.

Whatever the climate may be, the rainfall at any given place is highly erratic both as to time and quantity. And it has just been seen that it varies greatly with the locality. Two gauges not far apart may indicate different relative falls in different years, largely because of the varying directions of the rain storms. In tropical countries the fluctuations of the rainfall at any place are generally more violent than in temperate regions.

To obtain really reliable figures concerning the mean annual rainfall at any place, observations at that place should extend over a period of thirty to thirty-five years. The figures of the mean annual fall will then probably be correct to within 2 per cent. The degree of accuracy to be expected in results deduced from observations extending over shorter periods is as follows:

No. of years	35	30	25	20	15	10	5
Probable error	$1\frac{3}{4}$	$2\frac{1}{4}$	$2\frac{3}{4}$	$3\frac{1}{4}$	$4\frac{3}{4}$	$8\frac{1}{4}$	15

per cent.

These figures were deduced by Binnie[1] from an examination of rainfall figures obtained over long periods of time at twenty-six places scattered over the world. The errors may, of course, be plus or minus. They are the averages of the errors actually found and are therefore the errors which may, on the whole, be expected. But the extreme errors per cent. were as below and these, or even greater errors, may possibly occur.

No. of years		35	30	25	20	15	10	5
Extreme error	+	4.5	5.2	7.3	5.6	9	15	23
" "	—	4.7	6.9	9	9.2	12.5	16	30

It will be noted that the rainfall deduced from observations extending over a shorter period than thirty-five years is more likely to be in defect than in excess.

Binnie's figures also show that, at any place, the ratio of the fall in the driest year to the mean annual fall averages .52 to .68, with a general average of .60, and that the ratio in the wettest year to the mean annual fall averages 1.41 to 1.66, with a general average of 1.51. These figures are useful as a means of estimating the probable greatest and least annual fall, but they are averages for groups of places. The greatest fall at any particular place may occasionally be twice the mean annual fall. At some places in India, in Mauritius, and at Marseilles it has been two-and-a-half times the mean annual fall. The least annual fall may, in India, be as low as .27 of the mean. In England the fall in a dry year has, once at least, been found to be only .30 of the mean annual fall. The year 1921 was one of extraordinarily low rainfall in the south of England the fall being on the whole only .61 of the average. At many places the fall was .50 of the average.

The mean fall (average for all places) in the three driest consecutive years is, from Binnie's figures, about .75 and in the two driest consecutive years about .69 of the mean annual fall. The figures given above apply to all countries and to all places where the mean annual rainfall is 20 inches or more. In dry places the fluctuations are likely to be

[1] *Proc. Inst. C. E.*, Vol. cix.

greater. Also in places very remote from the sea; and in any place which lies between areas of high and low rainfall and which may accordingly be much affected by changes in the direction of the winds. At Kurrachee, with a mean annual fall of only 7.5 inches, the fall in a very wet year has been found to be 3.73 times, and in a very dry year only .07 times the mean annual fall.

In the British Isles the probable rainfall in the driest year may be taken to be, on the average, .66 of the mean annual fall. For periods of two, three, four, five and six consecutive dry years, the figures are .73, .77, .80, .82 and .835. Sometimes the figures are put rather higher than the above as for instance .67, .75 and .80 for one, two and three years respectively. Figures such as the above are much used in calculations for the capacity of reservoirs (Chap. IX., Art. 2). They are, however — as in other cases — only the average or most probable figures. At some places in the British Isles the figures for one year have been found to be from .59 to .64. In the United States the figures for one, two, and three years respectively may be taken to be about .60, .70 and .75 in the East and South, and .50, .60 and .70 in the North-West and plains. In the Rocky Mountains and Pacific coast area the figures are lower but unreliable because of the varied conditions [1].

The rainfall at any place may be below the average for a number of consecutive years, which may be as great as nine. The mean rainfall of such a group of years is likely to be about .82 of the mean annual fall. Similarly there may be periods of rain above the average, the corresponding figure being 1.20. In the 28 years, 1884—1911, the rainfall in south Lincolnshire was — except in the 9 years 1875 to 1883 and seldom for two of these in succession — below the average. In the Punjab a rough rule is that 1 year in 4 is a dry year.

Cycles, each covering several years of heavy or light rainfall, are irregular and cannot be predicted, though there is some evidence which tends to show that they recur after intervals of about 36 years.

[1] *Waterworks Engineering*, Turneaure and Russell.

When accurate statistics of rainfall are required for any work, the rainfall of the tract concerned must be specially studied and figures obtained for as many years as possible. Very frequently it is necessary to set up rain-gauges (Art. 5).

In the United Kingdom there are more rainfall records and the figures are more reliable than in most other countries, but there are still many areas in which there are few or no rain gauges. The British Rainfall Organisation [1] has full information as to such matters.

The area drained by a stream is called its "basin". It is bounded by a more or less defined ridge known as the "watershed". Of the rain which falls on any given basin a portion flows away and forms rivers and streams; it is called the "run-off". Another portion sinks deep into the ground and forms the "underground supply", and part of it subsequently appears in the form of springs. The "available rainfall" or "yield" of the basin is the sum of both the above. The stream whose basin is being considered is fed from both. A third portion of the rainfall is "lost" (see Art. 3). The bulk of the loss is that which evaporates and forms clouds or mist. The loss is the total rainfall minus the yield.

A "gathering ground" or "catchment area" is the same as a basin but the expression is usually applied to the local area whose drainage concerns some particular engineering work — whether designed or constructed — such as a reservoir formed by throwing a dam across a valley and intercepting the stream. In Great Britain the area of such a catchment is usually 3,000 to 6,000 acres, but it may be far greater.

Art. 2. Heavy Falls in Short Periods. When rain water, instead of being stored or utilised, has to be got rid of, it is of primary importance to estimate roughly — exact estimates are impossible — the greatest probable fall in a short time. This sometimes depends roughly on the mean annual fall. The maximum observed falls in twenty-four hours range, in the British Isles, generally from .05 to .10 of the mean annual fall — but on one occasion, shown in the accompanying table (Hanworth) the figure has been about

[1] Camden Square, London, W.

.33 — and in the tropics from .10 to .25, though the figure 40.8 given in the table, is only about .07 of the mean annual fall. Obviously this 40.8 inches could only occur in a place of high annual fall. The estimation of all heavy falls, and especially for periods shorter than twenty-four hours, depends chiefly on inference from other figures. Many such are given below and in Chapter VII. Art. 2.

Heavy falls of rain in short periods, in Inches

Period	India		E. Africa	United States			Penang	British Isles.
	Eastern Himalayas [1]	Bombay Hills [2]	Mombasa [1]	Californian Coast [3]	St. Louis Mo. near the Mississippi [4].	Texas, Rio Grande, Basin of 85 sq. miles [5]	George Town	
72 hours							20	
48 "				22.4	7.29		17	
30 "						12.5 (average over basin)		9.56 (Bruton, Somerset [7]).
24 "	40.8	19.5		11.5	6.85			8.27 (Hanworth near Norwich [6]).
11½ "								6 (near Norwich)
One night			10					
2¼ hours								4.7 (in London [8])
1 "	5			8.67				4 (in London)

[1] *The Times*, 11 Aug. 1922 and 20 May 1922.
[2] *Proc. Inst. C. E.*, Vol. CCVII. p. 407. Annual average fall 171 inches.
[3] *Waterworks Engineering*. Turneaure and Russell.
[4] *Engineering News Record*, Vol. 83, p. 814.
[5] *Proc. Am. Soc. C. E.*, Vol. 47, p. 445.
[6] *Paper on the Norwich Flood of August 1912*, Capt. H. Wood.
[7] *Rainfall of the British Isles*, Salter.
[8] *Proc. Inst. C. E.*, Vol. CCIV., p. 105.

The falls near Norwich occurred on 26th August 1912. Taking the usual rainfall day, beginning at 9 a.m., the highest figure was 7.31 inches, at Brundall. If notoriously wet places among mountains are excepted, there had been — in the 50 years over which records extended in 1912 — only one previous case (at Angerton in Northumberland) of a fall exceeding 6.5 inches in 24 hours. Dr. H. R. Mills states [1] that the chance of such a fall recurring in Norfolk is remote, and that out of 120 counties in Great Britain and Ireland, there are 17 in which the maximum recorded fall in a day is less than 3 inches and only 47 in which it exceeds 4 inches — in Norfolk the maximum had been 4.5 inches — but that it must now be recognised that 7 or even 8 inches in a day is possible in any county in either island (though at any place it may occur only once in several centuries) and that the fall to be provided for might reasonably be taken to be 4 inches; also that in a severe thunderstorm it might be 3 inches — or even more — in an hour, at any place in the centre or east of England, and something less than 2 inches in the west of Great Britain or in Ireland.

The following figures are given by Chamier [2] as applicable to New South Wales.

Duration of fall in hours	1	4	12	24
Ratio of fall to maximum daily fall	$\frac{1}{4}$	$\frac{1}{2}$	$\frac{3}{4}$	1

He considers that they are fair guides, erring on the side of safety, for other countries. They are probably fair average guides but they do not err on the side of safety. It has just been seen that the maximum fall in an hour at a place in the British Isles may possibly be not far short of the maximum recorded daily fall in the same locality.

The following falls have been observed in India.

The falls of 1 inch in ten minutes were frequently observed near the head of the Upper Jhelum Canal, a place where the annual rainfall is not more than 30 inches (see also Chap. VII., Art. 2). Sir John Benton, referring to this Upper

[1] *Proc. Inst. C. E.*, Vol. CCII. p. 284.

[2] *Proc. Inst. C. E.*, Vol. CXXXIV.

Period.	Fall. (inches)	Rate per Hour. (inches)	Remarks.
7 hours	10	1.43	
4.5 hours	7.5	1.7	
3 hours	12	4	Calcutta
2¼ hours	11.97	5.7	Upper Jhelum Canal.
2 hours	8	4	
1 hour	5	5	
20 minutes	1.6	4.8	Do. Do.
10 minutes	1	6	Do. Do.
5 minutes	.45	5.4	Do. Do.

Jhelum tract states [1] that "the Upper Jhelum Canal crosses formidable torrents from the Himalayas and from their offshoot the Pabbi range of hills. The rain-bearing clouds are pocketed and the precipitation is abnormally great".

The heaviest falls in short periods do not usually occur in the wettest years, and they may occur in very dry years. Nor do they always occur on a very wet day. Heavy falls may occur in dry climates.

Falls of rain of extraordinary intensity over very limited areas, and known as "cloud-bursts", occur in Lower California and in the drier parts of Upper California. They occur sometimes in England. No figures or accurate information concerning them are available.

Art. 3. Evaporation. Evaporation takes place from the surface of bodies of water, from the ground and from vegetation. Vegetation absorbs water, especially during the growing period, but the great bulk of the water absorbed is eventually evaporated from the foliage. All the evaporated water forms vapour, generally invisible, which rises up into the air and forms clouds. When these are condensed into water the "precipitation" of rain — or sometimes of hail or snow — takes place. The formation of dew is the

[1] *Proc. Inst. C. E.*, Vol. CCI pp. 35 and 36.

reverse process to that of evaporation and may give a negative reading on an evaporation gauge (described below).

Evaporation from the surface of reservoirs or other bodies of water depends on the temperature and dryness of the air. It goes on to some extent almost continuously. It is increased by wind especially if spray is caused. The depth evaporated at any particular spot in a given short period of time may vary greatly.

Evaporation from the surface of the ground depends not only on the temperature and dryness of the air and on the wind, but on the rapidity with which the rainfall runs off. On steep rocky ground the water runs off quickly. On flat porous ground it runs off slowly and more evaporation takes place. The evaporation does not chiefly take place directly from the surface. The rain sinks a short distance into the ground, and is subsequently evaporated. As the surface is dried the moisture from below is drawn up to it. When the surface and the soil immediately below it are quite dry, evaporation ceases. The losses by evaporation from the ground, by evaporation from the vegetation on it and by eventual absorption by the vegetation cannot be separated. Nor can they be measured. They are dealt with collectively and called the "loss" or the "loss by evaporation". The loss can be estimated in the manner explained in Art. 7.

In the British Isles the loss by evaporation in a year from reservoirs has been found to be as follows[1]: Staines 22.8 in. to 26.3 in., Dartmoor 16.6 in. to 21.3 in., Talla (Peebleshire, 904 feet above the sea) 13 to 22.9 in., and Derwent Valley (765 feet above the sea and generally cloudy) 10.25 in. to 19.62 in. In the last two cases the maximum loss was in 1911, a very sunny year. The loss in the four summer months — May to August — of 1911, was 3.6 in. per month at Talla and 3.03 in. in the Derwent Valley.

These losses somewhat exceed — as will be seen below — the losses by evaporation from the ground. The latter is of course intermittent though at times it is far greater than

[1] *Proc. Inst. C. E.*, Vol. CXCIV. pp. 3—152.

the loss from water. In countries where the bulk of the rainfall occurs in a few months and there are long periods in which the soil is dry but the water surface is still in existence, the annual evaporation from the latter is probably the greater.

In Australia and Tasmania [1] the loss by evaporation in a year from reservoirs has been found to be as follows: Sydney 37.4 in.. Melbourne 38.3 in., Adelaide 55 in., Perth 65.7 in. — the above are averages of many years' observations — Coolgardie 120 in., Brisbane 86.6 in., Laverton 180 in. In Queensland the figure is 120 in. in the dry western portion and 96 in. eastward of the range. In Tasmania the evaporation from Lake Sorell was 24.65 in. in 10 months. This was calculated from observations extending over 6 months.

The loss by evaporation from the surface of water in reservoirs may, in India and Egypt, be as much as .4 in. — in India even .5 — in a day in the hot season. Probably on such occasions a hot wind was blowing. The average in the hot season in India is about .3 in. daily. In the rainy season it is about .2 in. daily, in the cool season in Upper India .1 in. In the Bombay Presidency it has been found to be .2 in. daily for the eight dry months which include the hot and cool seasons [2].

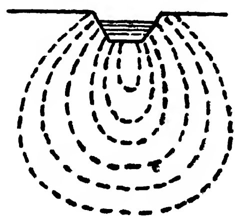
In the west of the United States the loss, as observed by the U.S. Geological Survey, ranges from 32.8 in. to 125.5 in. per year — except as noted below — the most usual figures being 50 in. to 100 in. and the highest figures being in California. Near the dry Salton Sea in California the evaporation was found to be 164.5 in. in a year and 22 in. in one month (July) which exceeds the figure for India [3].

In the Transvaal the evaporation [4] has been found to average 48 in. — 33 in. in the six months October to March — in sub-tropical Africa [5] 59 in., in Java [6] 73 in.

[1] *Proc. Inst. C E.*, Vol. CXCIV pp 74 and 133, and Vol. CCV. p. 363.
[2] *Indian Storage Reservoirs with Earthen Dams*, Strange
[3] *Irrigation Engineering*, Davis & Wilson.
[4] *Journal of South African Inst of Engineers*, Vol 18.
[5] *Proc. South African Society of Engineers*, Vol 16.
[6] *Proc. Inst. C. E.*, Vol. CXXVII p. 424.

At Konia in Asia Minor the loss from evaporation has been found to be 19.7 in. in the four warm and dry months[1].

For measuring the depth of water evaporated it is usual to employ a tank 3 ft. to 6 ft. square and 2.5 to 3.5 ft. deep. A gauge fitted with a vernier enables the depth evaporated to be measured. For this reason the evaporation from the latter is generally in excess in warm sunny climates. A tank used in Australia by the Meteorological Department was 3 ft. in diameter and 3 ft. deep and was jacketed, the outer casing being 4 ft. in diameter. The evaporation from a large reservoir tends to be greater than from an experimental tank placed in a position convenient for inspection, because of the greater wind sweep. On the other hand in the deep



reservoir the cold water keeps coming to the surface while the water in the experimental tank may become warm. Sometimes the tank is made to float on the surface of the reservoir. Otherwise it is placed on the ground. The evaporation from a very shallow reservoir is far greater than from a deep one.

Art. 4. Percolation and absorption. Percolation, also termed "seepage", consists in flow through the interstices of boulders, shingle, gravel or coarse sand. The flow is similar to that in pipes. Water percolating into the soil extends downwards and spreads outwards as it descends. None of it goes upwards. In fine sand and ordinary soil the interstices act like capillary tubes, the water is absorbed as by a sponge and it remains in the soil by virtue of capillarity.

[1] *Proc. Inst C. E*, Vol. CXCI. p. 364.

Owing to the combined action of capillarity and gravity the water spreads in the manner shown by the dotted lines in Fig. 1.

That part of the rainfall which sinks deep into the ground percolates through the porous strata and through the fissures in the underlying rocks. Its course is very far from being simple or regular. It may extend down to below sea level and may drive back the sea water which has also percolated down. Some of the underground water thus reaches the sea. The rest appears as springs where a porous stratum overlays an impervious one or where faults exist. The proportion which appears as springs is never the same in any two catchments.

If a deep excavation is made in the ground, the point at which water is met with marks the "spring level" or level of the "water table" or top of the "underground reservoir". This last expression does not imply that hollows of any size necessarily exist, or that the water table is level. The level of the water table in any particular locality is ascertained by observing the water levels in any wells which may exist, or it can be ascertained by boring. It is raised in a wet year. It is slowly but surely raised in the neighbourhood of any reservoir or canal which may be constructed and kept supplied with wator. Near Amritsar in the Punjab plains, the Upper Bari Doab Canal has caused the water table to rise some 50 feet in sixty years. Where it was 60 feet below the ground level it is now 10 feet below it. A dry year reduces the underground supply and the yield of the springs and wells. The supply may take long to replenish. In a flat river valley the level of the water table is generally — at places near the river — not very different from the water-level of the river.

The rate at which underground water travels depends, as in the case of flow in a pipe, on the gradient of the water table. Its movement is very slow. Observations by Slichter [1] show that with a gradient of 10 feet per mile the velocity

[1] *U. S. Geological Survey. Water Supply Paper* No. 140.

of flow is about 1 mile per year in fine gravel and far less in sand. It is lowest in compact clay[1].

The flow is affected by the temperature of the water. The lower the temperature the greater the viscosity or resistance to change of form of a mass of water.

When the water table is deep down, the rain-water at that place may never get down to it, the supply coming in horizontally. If just above the water table there is soil or fine sand it is kept moist, by reason of capillarity, for a certain distance up. This moisture, if the water table is not too deep down, may be joined by that which comes down from the surface. The flow of the underground water depends on the level of the water table and not on that of the moist soil above it, except in so far as this, if overcharged from above, raises the level of the water table.

When springs occur they are not necessarily visible or accessible. They may occur in the beds of streams or in reservoirs or lakes. In porous soil a stream may disappear and flow underground. If a stream whose channel is in porous material has the whole of its visible water diverted for any purpose, water may re-appear in the channel further down. In the case of a catchment area, the water which sinks into the ground may reappear in an adjoining catchment area, if for instance there is an impermeable stratum which slopes down towards the latter.

By observing the water-levels in wells, rivers or other bodies of water it is possible to construct a contour map showing the levels of the water table in a given area or district and to note the changes in its level. If a geological survey is also made, the information is far more complete and the directions of the movement of the water can be inferred and studied.

See also Chap. VI., Art. 2.

[1] Clay as ductile earth, has extremely small interstices and is nearly impervious to water Strictly speaking it is a mixture of silica and alumina. Practically any earth which, when kneaded up with water, is ductile and can be fashioned like paste, is called clay. Such clays vary greatly in their composition. Most earth is "loam" which contains sand, clay and vegetable and other matters

Art. 5. Measurement of rainfall. When the rainfall of a tract of country is to be studied it is frequently necessary, as already stated, to set up rain gauges. For a large area there should be one rain gauge for every thousand acres but it is not often possible to have so many. The chief difficulty is to get gauges read at remote places. These are usually on the highest ground and thus the records — at least in the British Isles — tend to give results which are too low. Some extra gauges may be set up for short periods in order to see whether the regular gauges give fair indications of the rainfall of the tract. If they do not do so some allowances can be made.

Sometimes there is only a year or two in which to collect figures. In this case the ratio of the fall to that, for the same period, at the nearest station where regular records are

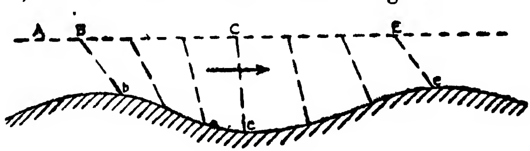


FIG. 2

kept, is calculated. This ratio is assumed to hold good throughout, and thus the probable rainfall figures for the new station can be obtained for the whole period over which the records have been kept at the regular station. In this way the rainfall records at Bombay were used by Sir Alexander Binnie for obtaining the rainfall at Nagpur, 600 miles distant, the rainfall at both places depending on the south-west monsoon.

The law of increase of rainfall with increase of elevation probably holds good in the case of several gauges along the bottom of a valley, or on several along a ridge, but the rainfall in the valley is probably greater than that on the ridge at the same elevation, that on the slope — between the valley and the ridge — at the same elevation having an intermediate value.

Suppose the wind to blow in the direction AE (Fig 2)

across a range of hills with fairly easy slopes and of rounded form. Just as in the case of flowing water, the general velocity of the wind will be greatest at the crest of the hill. The horizontal distance *bc* is less than BC. Similarly *ce* is greater than CE, or the intensity of the rainfall is reduced on a windward slope and increased on a leeward slope.

When there are obstructions such as bushes, walls, houses or trees, or abrupt changes such as cliffs or very steep hills, eddies are formed in the air just as in water. The effect of the eddies probably extends further to leeward of the obstruction or irregularity than to windward of it. Eddies may cause an excess or a deficiency of water to enter the rain gauge. A gauge should not be set up at any place where eddies are likely to occur. Regard should be had to the width of an obstruction as well as its height. In the case of a thick bush or broad leafy tree, twice its height should probably be the minimum distance to the gauge, in the case of a wall or long building three times its height.

A rain gauge should not be placed in a very exposed position — as many gauges are — because, with a high wind velocity, the rain gauge itself sets up eddies.

The differences — due to differences in elevation and other causes combined — between the readings of two gauges only 400 yards apart may be 20 inches [1]. In the case of the Derwent Valley the mean readings — over a period of 13 years — of gauges 8 and 9, both on the north-west side of the valley and 1,100 yards apart, were 61.3 and 46.8 inches, the elevations being 2,060 and 1,650 feet respectively.

The selection of proper sites for rain gauges requires care, judgment and experience. The best procedure in the British Isles is to consult the British Rainfall Organisation. When there is to be only one gauge its position should be chosen with special care.

An isohyetal line is one which passes through points where the mean annual rainfalls are equal. Such lines can be drawn with more or less accuracy when the rainfall has been obser-

[1] *Proc. Inst. C. E.*, Vol. CXCIV. p. 72.

ved for a number of years at many points in a large tract of country, and when this has been done the mean annual rainfall of the tract can be calculated. The area between each pair of consecutive lines is calculated and multiplied by the mean of the two rainfalls. But in a small tract such as the gathering ground of a reservoir, the number of points of observation is generally moderate and in order to draw isohyetal lines assumptions have to be made. In such cases it may be as accurate and very much easier — and it is quite common — to assign a given area to each gauge and assume that the rainfall at such gauge applies to that area. A still simpler plan — also common — is to take the average of all the gauges. Isohyetals, however, enable the variations in the rainfall to be seen at a glance.

In a tract of country in which the changes in the ground level are gradual and irregularities are slight, the drawing of isohyetal lines is not difficult; but in such a case the changes of rainfall from place to place are not likely to be great and the mean fall can be obtained by adding up and averaging the different falls at all the stations or even at a selected few of them. In the case of the Thames basin there were some 300 gauges, an isohyetal map was prepared, and the mean rainfall calculated year by year. The annual fall deduced from 24 selected stations was also calculated for six years and compared with the above. For whole years the difference was hardly appreciable. In comparing the rainfall for each month of a selected year a maximum difference of 9 per cent was found [1].

In a tract where the changes of level are abrupt and where there is much irregularity of the ground, the gauging stations must be more numerous. In the absence of numerous stations it is impossible to obtain the real mean rainfall whether isohyetals are attempted or not.

In the very common case of a tract in which there are comparatively few gauges, there may be none — or not enough — near the boundaries of the tract. In such a case the figures of any stations outside the tract should be ob-

[1] *Proc. Inst. C. E.*, Vol. CXCIV. p. 112.

tained and taken into account. Otherwise the rainfall at the boundary can only be guessed at. Any such lack of knowledge of course affects the calculations, whether isohyetals are drawn or not.

When there is a wind the drops of rain fall slantwise. This would not of itself cause any error. The top of the gauge is level. Any given area of level land receives the same amount of rain as when the drops fall vertically, provided the angle of incidence is the same at all points. But

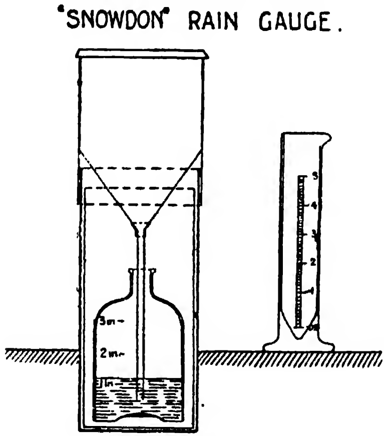


Fig 3.

the wind causes eddies and they affect the rain gauge. The wind velocity increases with the height above the ground. A gauge on a pole 25 feet above the ground has a "catch" of only 79 to 90 per cent of that at a height of 1 foot according as the wind velocity is 187 or 104 miles a day. The difference is greater the greater the wind velocity and the less the size of the rain-drops [1]. The velocity of the wind increases with the height above the ground, and so does the error of the rain gauge. Devices for getting rid of the eddies have been invented by Boernstein and Nipher [2], but they have not

[1] *Rainfall of the British Isles*, Salter.

[2] *Ency. Brit. Tenth Edition*, Vol. XVIII.

come into general use. If the rain gauge was sunk so that the top was level with the ground, rain falling outside the gauge would splash into it and vitiate the readings unless the gauge was surrounded by a trench.

There are several types of rain gauge. The top is circular with a knife-edge. In the British Isles the Snowdon gauge (Fig. 3) is the most common. The height above the ground is 1 foot and the diameter of the top 5 inches. In order that the gauge may be able to hold a considerable fall of snow, its top is not simply a cone but a cone surmounted by a cylinder 6 inches high. This long cylinder increases the bulk of the gauge and must increase the interference with the wind and consequently the eddyuing, just as it would in flowing water. In the United States Weather Bureau pattern, the height is 2 feet and the diameter 8 inches. The water is usually led by a tube into a narrow-necked glass vessel from which it is poured into a measuring glass. This plan minimises loss from evaporation and is suitable when the gauge cannot be read daily. In India it is generally read daily and there is a type of gauge 3 feet high, the very short cylinder at the top being connected — by a conical piece — with a long cylindrical container of smaller sectional area. If this is one-tenth of the area of the top, the depth of water — measured by a graduated rod — is divided by ten. On the Continent of Europe the heights of rain gauges are 1 to 1.5 metres.

In the case of a rain gauge near George Town, Penang, where the fall in a day was likely to be excessive, an additional storage tin for the water had to be provided. It was covered by a roof to reduce evaporation [1].

A self-registering gauge can with advantage be used under many circumstances. One in use in India is used as a check on the ordinary gauge. There are two buckets, each on the arm of a lever. One bucket or the other is always under an orifice through which the water passes down. When a bucket fills it descends and empties and the other bucket comes up. Mechanism actuates rows of figures which are presented

[1] *Proc. Inst. C. E.*, Vol. CXCIV. p. 87.

at a small window. Now and then the gauge gets out of order but is on the whole fairly satisfactory. There are other kinds of sel-registering gauges, generally somewhat liable to get out of order.

To the engineer studying the rainfall of a tract of country — especially in connection with floods — a knowledge of the intensities of the various falls of rain is of great value. Such information can be obtained by means of special gauges but they are expensive and contain clockwork which is apt to be unsatisfactory. A gauge without clockwork has been devised by Meares [1]. The tube which conveys the water from the funnel discharges a fine jet in a horizontal direction. The greater the intensity of the rainfall the greater the distance to which the jet springs. A number of containers are arranged to receive it, each container corresponding to a given degree of intensity. The instrument thus gives the daily amounts of rain corresponding to the different intensities. If the times and order of occurrences of the different intensities are required a clock-driven drum has to be added.

Sometimes buildings or other obstructions come into existence after a gauge has been long in use and they may not be reported and may to some extent vitiate the readings.

Where heavy falls of snow are likely the gauge should be read daily. Otherwise the whole of the rainfall is not likely to be registered; and a height of 1 foot above the ground for the gauge is not enough. When the gauge is not read daily, a moderate fall of snow collected in the gauge may melt and so be measured, but a heavy fall may overflow. In such a case the depth of snow on the ground can be measured in a sheltered place and 12 inches of snow considered as equal to 1 in. of rain.

The best general check on the readings of a gauge is comparison — say weekly — with other gauges.

For computing rainfall, the day in Great Britain is from 9 a.m. to 9 a.m. and this period should be adopted, especially

[1] *Selected Engineering Papers*. No. 2. Inst. of Civil Engineers. 1923.

when any specially heavy fall extending over parts of two days is being considered.

Art. 6. Influence of forests and lakes. In a forest some of the falling rain is intercepted by the foliage — supposing it to be present — and is thus in a favorable position for rapid evaporation. Light rain may fail altogether to reach the ground. But generally — and especially in hot countries — a greater effect of the foliage is to shield the ground from the rays of the sun, and so to reduce evaporation and thus, on the whole, augment the yield. Forests in sunny climates tend to reduce the temperature, and they may thus increase the actual rainfall to some extent.

More important than the above is the effect of forests on floods. In forests, the *humus* or mould formed from leaves, etc., absorbs and retains moisture. It acts like a reservoir, so that the run-off takes place slowly and the denudation and erosion of the soil is checked. The roots of the trees or other vegetation also bind the soil together. Forests thus mitigate the severity of floods and reduce the quantity of silt brought into the streams.

The Adviser on Irrigation and Flood Prevention in the Malay States has recently reported that the conversion of dense vegetation into plantations with clean weeded ground, has enormously increased the rapidity of the run-off in heavy storms of rain. A similar report has been made by the Conservator of Forests in the Central Provinces of India. Measures to put a stop to the destruction of forests or reforest bare land may enter into questions of the régime of streams or the supply of water. On the Rhine, and in other districts, increase in the severity of floods has been distinctly traced to deforestation of the drainage area.

Shrubs, vegetation and crops act in the same way as forests, but with less effect. Generally the greater the bulk of the growth the greater its influence. The effect of ploughing the soil is to reduce the run-off, but increased population is accompanied by better drainage of the land and this increases the run-off.

The rainfall may be greater on lakes than on the neigh-

bouring country. After the construction and filling of nine square miles of lakes (reservoirs) in the Bombay hills, it was found that the rainfall in the catchment area — 22.1 square miles, see Fig. 63, Chap. VI — was affected. The rainfall on the hill tops had been, on the average of 5 years, some 17 per cent greater than the mean of the whole area. The two became (average of 6 years) nearly equal [1].

Art. 7. Yield of rainfall. The best method of ascertaining the yield from a catchment area is to observe the discharge of the stream regularly for a long period, but when the question of utilising a given catchment comes up, there are probably few or no records of such discharges. Some rainfall records generally exist. The discharge is calculated from the rainfall. The catchment has to be surveyed to ascertain its area. Its character has to be studied. The probable loss of rainfall is estimated from actual losses which have beenf ound to occur elsewhere. Hence the necessity for recording, publishing and discussing such losses. At present a vast amount of information which is collected in reports, does not see the light.

In the British Isles and in other European countries, the discharge measurements made may be quite insufficient even after the work has been decided on or while it is in progress. The reason for this is the expense attendant on the construction of weirs and the objections of landowners to such works. In any case there may be difficulty in observing the volume which goes to waste during floods. When works for utilising the flow have been constructed, discharges should always be observed. The discharge figures are more reliable when a gauging weir is constructed than when the discharge is computed by noting the rise and fall of the water in the reservoir and the quantity passed through the pipes to the place of consumption. This last quantity can, however, be accurately measured if there is a Venturi water meter. Where a weir has been constructed, careful and continuous observation of the discharge should be made. Whatever the form of the weir, its coefficients

[1] *Proc. Inst. C. E.*, Vol. CCVII. p. 408.

should be ascertained, at least approximately. In the United States the rainfall records are generally defective but far more discharge observations are made than in most countries.

In the case of a river of considerable size the area of the basin is large and much of it free from great irregularities, so that the number of rain gauges in the basin may — at least in the British Isles — be enough to afford a fair indication of the average rainfall. The average river discharge may be known, having been deduced from discharge observations. The ratio of the yield to the total rainfall depends chiefly on the nature and steepness of the surface of the catchment area, on the temperature and dryness of the air, and on the amount and distribution of the rainfall. The ratio is far greater when the falls are heavy than when they are light. A low annual fall may however give a good percentage of run-off if the fall is all concentrated in a few months. Again, when the ground is fairly dry and the temperature high — as in summer in England — nearly the whole of the rainfall may evaporate; but when the ground is soaked and the temperature low — as in late autumn and winter in England — the bulk of the rainfall runs off. A surface baked hard by the sun does not, however, absorb much water at first. When moderately moist it absorbs more, when saturated least. After a long period of hot dry weather, even a heavy fall of rain may have little effect on the streams. When the ground is frozen hard the rain which first falls on it nearly all runs off. In the eighteen years from 1893 to 1900 the average discharge of the Thames at Teddington, after allowing for abstractions by water companies, was in July, August and September 12 per cent. of the rainfall — 6.9 inches — in its basin, and in January, February, and March 60 per cent, of the fall which was 5.9 inches. The total fall in the year was 26.4 inches. Some rivers in Spain discharge, In years of heavy rainfall, 39 per cent., and in years of light rainfall 9 per cent. of the rainfall.[1]

Rankine gives the ratio of the yield to the whole fall as

[1] *Proc. Inst. C. E.*, Vol. CLXVII.

1.0 on steep rocks, .8 to .6 on moorland and hilly pasture, .5 to .4 on flat, cultivated country, and nil on chalk. These figures are approximate and average. The loss of rainfall is not proportional to the rainfall. The practice of taking it to be so is, however, still adopted in many cases in America and in India. It is far more correct to consider the loss as a fairly constant quantity in any given locality, but increasing somewhat when the rainfall is great. The yield of rainfall in Great Britain has frequently been overestimated. Sometimes it used to be taken as .60 of the whole fall. The loss is now usually taken to be 12 to 15 inches in high moorland where the rainfall is 40 to 80 inches, the loss being greater the higher the rainfall. In ordinary country where the rainfall is 20 to 40 inches the loss is taken to be 12 to 20 inches, being, as before, greater the higher the rainfall. All the above figures are, however, general averages. The proper estimation of the yield at any place in any country, depends on experience and judgment and on the extent to which figures for actual cases of similar character are available. Regarding the run-off from saturated land during short periods, see Chap. VII., Art. 2.

The accompanying statement gives information concerning the basins of some rivers [1]. In the Thames basin the loss — 19 inches — calculated from the table, is far better determined than the loss — 21 inches — in the case of the Weaver. The second table gives some figures [2] for catchment areas of smaller sizes than river basins. The British areas were of the usual high moorland type.

The Derwent Valley figures — obtained by Sandeman — are specially reliable and are probably better than any similar figures previously obtained in the United Kingdom, the sites for its rain gauges having been numerous and fixed by experts and the discharge measured by a weir. The mean

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[1] *Proc. Inst C E.*, Vol. CLXXXVIII p. 283 and Vol. CXCIV. pp. 3—152. *Project Estimate for Irrigation Works, Central Siam*, Vol. II., Bangkok, 1915.

[2] *Proc. Inst. C E.*, Vol. CLXXXVIII. p. 283, Vol. CXCIV. pp. 3—152, Vol. CCVII. pp. 24—120, 386—404.

River	Catchment Area (square miles)	Mean annual Rainfall (inches)	No. of years over which observed	Yield (per cent. of fall)	Remarks
Murrumbidgee River above Gundagai, Australia	8,300	28.6	15	21.6	
Darling River above Wilcannia, Australia	235,000	11.22 to 26.81	9	0.65	
Murray River at Morgan, Australia	408,000	15.56	8	2	
Nepean River, N. S. W.	284	44.3	1	44	Bare, broken ground
Cataract River, N. S. W.	70	54	1	45	Do. do.
Thames, England		26.9	21	30.1	
Weaver, England	544	29.46	2	28.4	
Nile		32.5		4.26	
Seine		29.4		27.8	
Rhine		32.6		44.2	
Neva		20.9		70.2	
Potomac, U. S. A.	11,043	45.5	5	53	
Near King William's Town, Cape Colony	105	27		21	Hills with forest and bush
Menam River, Siam	43,320	49	9	22.1	Plain of Central Siam.

annual rainfall was 61 inches at the higher end of the catchment and 35 inches at the lower.

In the Farg, Slateford and Gameshope areas the discharges were also obtained by weirs. The Gameshope area formed part of a larger one (Talla Water, 6,180 acres) but the discharge from the remaining portion seems to have been unreliable, one rain gauge — there were four — having

Locality	Catchment Area (acres)	Elevation above sea-level (feet)	Rainfall: No. of years over which observed	Rainfall: Mean Annual Fall (inches)	Rainfall: Loss (inches)	Rainfall: Yield (per cent of mean)	Rain Gauges: No.	Rain Gauges: Acres per gauge	Remarks.
Derwent Valley Derbyshire	31,288	540 to 2060	13	47.8	11.8	75.2	46	700	Considerable areas of peat. Shales and sandstones.
Glencorse Burn, Midlothian	3,823	750 to 1750	7	40.5	16.6	59	4	956	Igneous Rocks. In Farg area much boulder clay overlaying the rock.
Slateford Burn, Perthshire	1,166	650 to 1200	3	37	13.8	63.7	3	389	
River Farg, Perthshire	1,437	550 to 1000	3	35.6	10.5	70.5	4	359	
Gameshope, Peebleshire	3,000	950 to 2600	7	68.4	14.1	79.3	4	750	
Elan Valley, Wales			1	60.9	17.5	71.3	22		1908.
			1	63.4	16.2	74.4	22		1909.
			1	64.5	21.3	66.9	22		Hot dry summer, 1911.
			1	82.3	19.6	76.3	22		Dull wet summer, 1910.
Plymouth Waterworks, Dartmoor	4,885		6	54.9		generally over 100.			Much decomposed granite. Water comes in from adjacent areas.
Langsett, near Sheffield	5,203	1037 to 1516	2	51.9	13.9	72 6	6	867	
Loxley Valley, near Sheffield	10,725	650 to 1326	2	43.5	15.2	64	7	1532	
Redmires and Rivelin, near Sheffield	4,978	700 to 1300	2	42.5	13.6	66.7			Bed of clay over a large portion.
Blackburn Waterworks, West Riding of Yorks	6,000	550 to 1730	26	63.4	6.7	89.4	6	1000	Millstone grit series with outcrops of shale and in places, limestone.
Newport, Isle of Wight				32	19.2	40			
Tansa Lake, Bombay	33,600	362	9	85.7		51			Flat country. Heavy falls in rainy seasons.
Vehar and Tulsi Lakes, Bombay	5,614	280	9	82.2		72			Steep, much rock.
Shirawta and Walwhan Lakes, Bombay	10,586	2123 (average	4	100		70	29	365	Steep hills. Figures in column 7 are estimates based on several actual observations.
				175		80			
				250		90			
Nagpur, Central India	4,224	Gently sloping and open.	1	6.8		5			The rainfall shewn is that of the rainy season, June to October. In the rest of the year the rainfall averages only 2 or 3 inches.
				19.5		16			
				29.8		27			
				39.3		40			
				53.7		40			
Hong Kong Waterworks	1,509			8.3		66 to 75			
Near Cape Town, Cape Colony	22,080	2500		43		40	26	850	Bare hills. Rainy season only. (May to October).
Do. Do.	110	2500		31.5		51			Do. Do. Do.

been seriously affected by snow. It remained frozen every year for months and was only read once a month. Also the discharges running to waste in floods were imperfectly known. The figures for this remaining portion showed, in most years, little or no loss of rainfall and sometimes a gain.

The variation in the yield from year to year is generally greater the smaller the catchment area.

The yield from any catchment area for a short period, e.g. a month, if calculated from the rainfall for that month may of course be wrong, because of the flow of springs. Sometimes it is right when calculated from the rainfall for a month previous to the one in question and separated from it by a given interval of time.

In countries such as India where there is a monsoon or rainy season and little rain at other times, so that the catchment is always dry at the commencement of the rainy season, it is usual to take account of the rainfall of the wet season only, as in the case of the Indian figures given in the above table, the rainfall in the rest of the year being considered as giving no yield.

In the case of Nagpur the figures are all for the same year. Each pair of entries, in columns 5 and 7, shows the total rainfall and yield up to a particular date, the first entries being for 30th. June. The figures thus show yearly results for imaginary years in which the monsoon ended at various dates.

When the whole rainfall of the year consists only of occasional and not very heavy falls so that the catchment is nearly always dry, the yield of each fall depends so much on the state of the catchment when the fall begins, that general estimation of yield is very difficult. The yearly yield may vary from 10 per cent of the rainfall to nil. The following is an example, the figures being those of the Helena River basin (569 square miles) in Western Australia [1].

In studying the conditions of a catchment area it is often convenient to prepare an accumulative diagram, The ab-

[1] *Proc. Inst. C. E.*, Vol. CCV. p. 354.

Year	Rainfall (inches)	Yield (per cent. of rainfall)	Remarks.
1914	14.6	.80	
1902	19.3	.22	
1911	21.5	2.18	
1897	24.5	34	
1908	24.9	2.43	
1901	25.0	69	
1912	26.1	2.94	The year of lowest rainfall is entered first, then the next lowest and so on.
1897	27.2	83	
1906	27.6	3.80	
1913	28.5	2.38	
1898	30.2	1.50	
1903	30.3	2.50	
1909	32.0	3.83	
1904	32.7	3.20	
1900	33.2	3.50	
1910	34.5	7.81	
1905	35.1	7.20	
1907	37.7	7.54	
1915	42.4	5.30	

scissæ are times as in an ordinary hydrograph, but the ordinates for any date show the total rainfall and total loss — or total yield — up to that date, as in the case of the Nagpur catchment area in the preceding table.

During periods of drought the streams are fed from springs and the volume of flow grows steadily less. The minimum discharge or "flow" is a matter of the greatest importance. It shows the rate at which a reservoir can be supplied after a drought has been some time in progress. The minimum flow — per thousand acres — in the Talla area was .59 c. ft. per second, in the Glencorse area .25, in the Slateford .16 and in the River Farg. .085, each figure being the average for a week. In the Loxley Valley the minimum flow was .22 c. ft. per second, in the adjacent

Redmires and Rivelin area .15 c. ft. per second, each figure being the average for a month. In the Plymouth area the minimum flow (1 week) was 1.09 c, ft. per second, in the Blackburn area .59 c. ft. per second. In the Derwent Valley the minimum flow was .29 c. ft. per second in one day. On the eastern side of the Pennines .25 c. ft. per second is fairly common. The figure for a month was on the average about 1⅓ times as great as that for a week; and the figure for 4 months about double that for a month. The figures for the Blackburn area were obtained in 1887, all the others in 1911.

Figures for the extreme dry-weather flow in different catchment areas in the Vyrny gathering ground, N. Wales, are as follows. In 11 catchments the areas are from 780 to 5,420 acres and the discharge per 1,000 acres .153 to .650 c. ft. per second. In 4 larger cartchments — 13,369 to 34,259 acres — the discharges vary from .067 to .204 c. ft. per second.

The time during which the flow is much below the average is also a matter of great importance. Droughts usually occur at long intervals so that observations to ascertain the figures must extend over long periods. The minimum discharge in hilly country in temperate climates averages about .2 c. ft. per second per square mile. In countries with long dry periods, the small streams may dry up completely.

On mountains when snowfall takes the place of rainfall, the snow accumulates and while the cold continues none of it runs off. It forms a reservoir and when the weather becomes warmer the snow melts and furnishes a supply of water independent of the rainfall. Drifted snow of course melts slowly. When rain falls on snow the latter may melt rapidly and a very heavy run-off occur. In the cases of streams fed from the melted snows of the Sierra Nevada in California, attempts have been made to forecast the discharge by observing the snowfall in a selected region and assuming that its ratio to the normal is the same as that of the stream discharge ratio. Another method is to make surveys of selected parts of the snow-fields, the depth

of snow being measured. Uniformity of snow cover in adjoining stream basins is a usual condition and it is therefore practicable to predict the discharge from large snow-clad areas [1].

In order to show the ratio of the discharge at one point of a stream to that at another point, a diagram has been prepared by Dennis [2]. The river Kern is a typical Californian stream in mountainous country. At the "First Point of Measurement" the discharge has been observed since 1894. At Fairview, 75 miles higher up the stream where a hydro-electric generating station was being constructed, the discharge has been observed since 1912.

The ratio of the discharge at Fairview to that at First Point varies with the time of the year. It is highest in July and lowest in January. The differences are due to climatic conditions. In July there are occasional storms at the higher altitudes, in January rain at the lower altitudes. The ratio under discussion fluctuates in an irregular manner, but by assuming that the average ratio must vary in a regular and steady manner from month to month, regular curves were got out. The ratio was for July about 55/60 and for January about 20/60, the maximum discharge at First Point ranging from 800 to 4,000 cubic feet per second. For small discharges the ratios tend to become more nearly equal. The ratios appear to be liable to an error of some 10 per cent

Figures such as the above, for any stream, will be useful in preliminary investigations concerning the discharge of any other stream whose conditions are nearly similar.

The yield from a catchment area may be greatly increased by making furrows or small drains to lead the water into the streams [3]. Field drains — subsoil — commonly used in England have the same effect. (See also Chap. VII., Art. 1).

If a large reservoir exists in the catchment, its area should be deducted when the yield from the catchment is calculated.

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[1] *Engineering News Record*, Vol. 86, pp 244—248 and 300—304.
[2] *Proc Am Soc. C. E.*, Vol 84, pp. 551—566
[3] See Chap. VII., Art. 1.

The whole rainfall on the reservoir should be added and allowance made for losses from the reservoir.

Even when there are weirs or other arrangements for observing the discharge they are frequently unable to deal with all the water in floods. The flood discharge has often to be roughly estimated and this reduces the accuracy of the figures obtained.

In the case of any portion of a catchment lying at a great elevation, the loss from evaporation is likely to be below normal owing to the lower temperature.

In an ordinary catchment area in the British Isles the flow in the evening is usually some 10 or 12 per cent less than in the morning owing to greater evaporation during the day.

CHAPTER III

THE ACTION AND TENDENCIES OF STREAMS

Art. 1 Silt. Silt is carried in suspension in the water of a stream or is rolled along the bed. The quantity of silt suspended in each cubic foot of water is called the "charge" of silt. The eddies wich are constantly thrown off from the bed keep it in suspension.

Silt generally consists of sand and mud[1] but either of them may be in excess of the other to any extent, or be absent. The larger a grain or particle of any kind of silt the greater is the rate at which it sinks in still water.

Silt deposited in a channel anywhere near its off-take reduces the discharge. At other points it reduces the capacity of the channel and thus raises the water-level. This, in the case of a canal, may endanger the banks or cause increased loss of water through absorption. The frequent removal of silt is expensive. When silt is sandy it does harm to fields, and is extremely injurious to machinery. It may also do damage to pipes. Silt tends to fill up reservoirs, though the process in generally slow. On the other hand, it tends to render canals and reservoirs watertight and assists very greatly in the formation of berms and in the training of channels. When fairly free from sand it is highly beneficial to fields. Silt gives much trouble on the irrigation canals of India, Egypt and Mesopotamia, and some trouble in the Western States of America, but in most of these cases the trouble is largely — or wholly — set off by the advantages mentioned above.

Mud exists in nearly the same proportion near the surface and near the bed. Sand — at least coarse sand — is oftener

[1] Not necessarily sand and "clay". See Chap. II, Art. 4.

rolled than carried, When carried it is usually in much greater proportion near the bed The distribution of silt in any particular stream can only be ascertained by observation, or by experience of similar streams. It is a matter of practical importance, as affecting the best bed level for a branch taking off trom the stream. The results of observations show considerable discrepancies, even when averaged, and individual observations very great discrepancies. In some rivers 10 to 17 feet deep, the silt charge has been found to increase at the rate of about 10 per cent, for each foot in depth below the surface. In others, with depths ranging up to 16 feet, the silt charge at about three-fourthe or four-fifths of the full depth has been found to bear to that near the surface, a ratio varying from 1¼ to 2.

The quantity of silt in water is found by taking specimens of the water and evaporating it or, if the silt is present in great quantity, leaving it to settle for twelve hours — an ounce of alum can be added for every 10 cubic feet of water to accelerate settlement — drawing off the water by a syphon, and warming the deposit to dry it. The deposit is then weighed or measured, If filled into a measure, the volume may depend greatly on the manner in which it is filled in. Thus many of the figures of volume given below may be taken to be approximations

For making comparisons, weighing is by far the best method, but as regards engineering considerations the volume is the figure chiefly needed. When silt deposits in a channel or reservoir, the effect which it produces depends on its volume. When there is heavy silting or scour in canals, the volume — when required accurately — is found by taking careful cross-sections of the channel.

Sandy silt, when a canal is laid dry, soon becomes more or less dry, It does not crack in drying, Mud does not dry so quickly and sometimes remains wet, and sticky for a very long period, When it dries it contracts and cracks

A few experiments have been made in order to ascertain the dry weight and dry volume of a cubic foot of silt deposited in a canal or reservoir. The weight of a cubic foot

of wet silt — probably in about the same state as that in which it would be when lying submerged in a canal or reservoir — was found to be from 89.5 to 104.7 lbs. It averaged about 97 lbs.[1] when the silt was sandy and 95 lbs.[2] when it was entirely mud. The weight of water in the cubic foot was about 13 lbs. in the case of sandy silt and 45 lbs, in the case of mud, the dried material weighing about 84 lbs.[3] in the case of the sandy deposit, and 50 lbs.[4] in the case of the mud. The specific gravity of the dried sandy deposit is given by Davis and Wilson as about 2.60. Perhaps 1.60 is meant. Even for pure sand the figure is only about 2. Rothery mentions 2.65 as the specific gravity of silt which appears to have been wet. This figure also appears to be highly erroneous. In the Tigris experiments by Lewis (Art. 2) some pieces of the dry silt (mud) were solid and free from cracks. Such a piece weighed 108 lbs. per cubic foot. Molesworth gives the weight of sand as 125 lbs. per cubic foot, clay 126 lbs. and mud 104 lbs.

If in two streams the percentages — of silt to water — by weight are the same, the percentages by volume will differ unless the two silts are identical in constituents and proportions. No rule can be given as to the ratios to one another of the weight percentage and the volume percentage. The weight percentage is always the greater. Comparative figures for the Irrawaddy are given below.

Sand can be classified by observing its rate of fall through still water. A sand which falls at .10 feet per second is, in India, called class $\overline{.1}$, and mixed sand which falls at rates varying from .1 to .2 feet per second is called class $\frac{.1}{.2}$. Fig. 4 shows a sand separator designed by Kennedy. The scale is $^1/_8$. It has a syphon action, and the rate of flow can be altered by altering the length of the exit pipe. Suppose it is desired to measure the sand of class $\overline{.10}$ and all heavier kinds. The pipe

[1] *Irrigation Engineering* Davis and Wilson. Buckley mentions 98 lbs as the weight on the river Kistna

[2] *Irrigation Pocket Book* Buckley

[3] 86 lbs according to Rothery *Proc Inst. C E.* Paper No 4416, "An Irrigation Project of the Californias"

[4] 53 lbs in the case of the Rio Grande mentioned below.

is adjusted so as to give a velocity of .1 foot per second to the upward flowing water, which then carries off all silt finer than class $\overline{.10}$ All heavier silt falls into the glass tube. It can be separated again by being mixed with water and

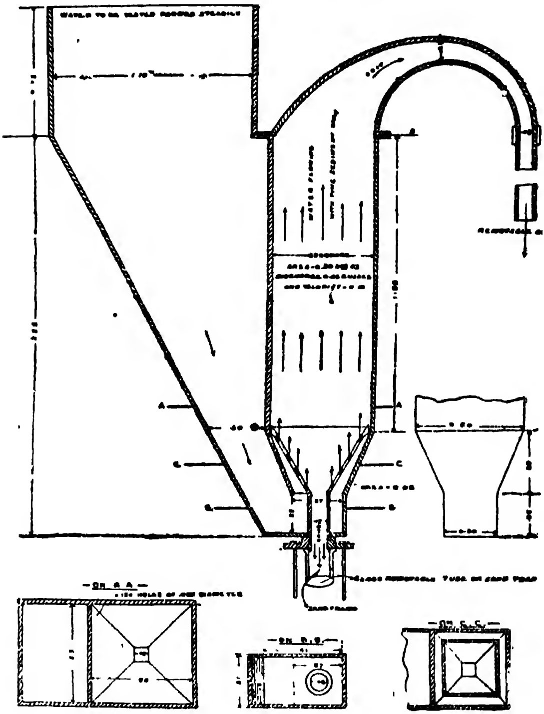

FIG. 4.

passed through the instrument again, the velocity of flow through the instrument being increased.

The quantity of silt present at various depths in a stream can be found by pumping specimens of water through pipes. At each change of depth the pipe, delivery hose, etc., should be cleaned, Allowance must be made for the velocity of ascent of the water up the pipe. Suppose this to be 1.4 feet per second.

Then the velocity of sand of class .2 would be 1.2 feet per second, and the quantity of sand actually found in the water would have to be increased by one-sixth. In the case of Buckley's experiments in Egypt, referred to below, only small quantities of water could— owing to the cost of transport to the laboratory — be dealt with. Pumping was not considered safe because of the chance of some of the silt being deposited in the pipes or hose. The silt was obtained by letting down a bottle in a cage to the desired depth and then extracting the cork by means of a cord previously attached to it.

Rolled material is very seldom measured. It can, no doubt, be trapped in a box or orifice, as suggested by Kennedy. It may be the most important item and give far more trouble than the suspended silt. Sand-traps are mentioned in Chap. IV., Art. 1. On the Exe, the bed, above a weir, was kept at a constant level for a long period by dredging, The quantity dredged was measured, The object was to ascertain the volume of the rolled material [1].

The quantity of silt suspended in water varies enormously not only in different rivers but in the same river from day to day. Any figures which can be given are merely rough indications of what may occur. The causes of this will be stated below. The quantity is greatest during floods.

In the river Tay, near Perth, the silt was found to be ordinarily $\frac{1}{10,000}$ of the volume of water, and at low water only $\frac{1}{28,000}$. In 26 English rivers the suspended solids were found to be, by weight, from $\frac{1}{25,000}$ to $\frac{1}{4,202}$ of the water. When the proportion was less than $\frac{1}{8,000}$ the rivers were classed as "clean". The samples of water were taken from the surface at ordinary stages of the flow and not in floods. In the Ouse near Huntingdon the proportion was $\frac{1}{100,000}$. [2]

The proportion of suspended silt to water by weight — during floods — has been found to be, on the Tigris $\frac{1}{53}$, on the Rhine $\frac{1}{100}$, on the Indus $\frac{1}{237}$, on the Po $\frac{1}{300}$, on the Kistna $\frac{1}{302}$, on the Nile $\frac{1}{670}$, on the Ganges at Hardwar — shortly

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[1] *Proc. Inst. C. E.*, Vol. CXCIV. p. 130.
[2] *Proc. Inst. C. E.*, Vol. CCII p. 280.

after leaving the Himalayas — $\frac{1}{700}$, on the lower Mississippi $\frac{1}{1,500}$ on the Irrawaddy $\frac{1}{1,760}$.

In the Irrawaddy observations, made by Samuelson, the weight of water dissolved (not suspended) in the water was also ascertained. It varied from of that of the water. The suspended silt varied from $\frac{1}{1,126}$ to $\frac{1}{3,740}$ and had no particular relation to the volume of the flood. In the case of sand on the Irrawaddy $\frac{1}{590}$ by volume was found to be equivalent to $\frac{1}{370}$ by weight [1]. The sand was passed through sieves with the following results.

Number of wires or threads to an inch	Percentage of sand stopped by sieve	
	Irrawaddy at Mandalay	Chindwin at Kindat
10	0	5
16	4	10
30	19	50
40	13	16
75	34	19
100	10	
over 100	20	
Total . .	100	100

Indian river sand has sometimes been roughly classified as very fine, fine, medium, coarse and very coarse when only 25 per cent. of it is stopped by sieves, having respectively 100, 80, 60, 40 and 16 wires to an inch. There are very great differences in the degrees of coarseness. In Buckley's experiments sand which was stopped by sieves having 100 and 200 meshes to the inch was classed as coarse or fine respectively. Material that passed the smaller mesh was classed as "fine silt and clay". The sand on any river bed becomes

[1] *Proc. Inst. C. E.*, Vols. CCII. p. 231 and CCIII p. 362. *Note on the Irrawaddy River.* (Government Press, Rangoon).

finer and finer the further from the source. The sand on the bed of a canal also becomes finer the further from the head. See also Notes on Sand at end of Chapter VIII.

Observations made by Chatley [1] on the tidal river Huangpu in China, show that the proportion of silt was generally less than 1 in 1,000 by weight and that the heavier part of the silt settled at the rate of about a foot in an hour — the diameters of the particles being about $\frac{1}{10}$ th of a miliimetre — and the finer portions occupying 3 days to a week in settling.

Silt observations on the Rio Grande at San Marcial, New Mexico, extending over sixteen years, give an average proportion of silt by volume of $\frac{1}{60}$. In one year the proportion was $\frac{1}{25}$. These proportions are extraordinarily high. The silt was all mud [2].

In the river Sutlej at Rupar, near where it issues from the Himalayas, the silt in the flood season is heavy. Out of 360 observations, made at various depths during the flood seasons of four consecutive years — ending with 1897 — in water whose depth ranged up to 12 feet, the silt was once found to be 2.1 per cent. by weight, of that of the water It was more than 1.2 per cent, on four occations, and more than 0.3 per cent. (or 3 in 1000) on sixty-four occasions. Generally about half of the silt consisted of mud and of sand of classes finer than $\overline{.10}$, about one-third was sand of class. $\frac{.1}{.2}$, and the residue was sand of class $\frac{.2}{.3}$.

In the Sirhind Canal, which takes off from the Sutlej at Rupar, the maximum quantity of suspended silt observed in the four flood seasons was 0.7 per cent., on one occasion out of 270, and it exceeded 0.3 per cent. on twenty-five occasions. About 80 per cent, of the silt was mud. The silt was much less than in the river water because of the steps taken to exclude the coarser silt (Chap. IV., Art. 2). The silt suspended in the canal water averaged, during the whole of one flood season —1898— about $\frac{1}{1,700}$ of the volume of the water. This would be perhaps $\frac{1}{1,200}$ by weight. This suspended matter appears to have been the sand. The mud was not measured.

[1] *Proc. Inst. C. E*, Vol Vol. CCXII. p. 400.
[2] *Engineering News*, 1st Jan. 1914.

The silt deposited on the bed of the canal, in a period of a few days, was sometimes as much as $\frac{1}{1,000}$ of the water which had passed along, and occasionally as much as $\frac{1}{500}$. It was nearly all sand, only about 3 per cent. being mud. Sand of classes finer than $\overline{.1}$ gave no trouble, and were to be eliminated in future investigations.

Deposits taken from the beds of the river and canal respectively were found to contain the following percentages near Rupar and at places lower down

		Class	$\frac{\overline{.2}}{5}$	$\frac{\overline{.1}}{\overline{.2}}$	$\frac{\overline{.00}}{\overline{05}}$
River Sutlej	Rupar		10	36	54
	180 miles lower down . .		3	6	81
Sirhind Canal	Miles 2 to 5		40	55	5
	50 miles lower down . . .		15	57	28
River Chenab, Khanki			19	42	39

Art. 2. Silting and Scouring Action. The theory of silting and scouring action is fully discussed in *Hydraulics* The main practical points are as follows.

In any stream the "full charge" of any particular kind of silt is the greatest charge which it can carry, taking the average from surface to bed For a given value of V the full charge is less as the depth, D, is greater. The silt-transporting power of a stream is the sum of its carrying and rolling powers.

The greater the size of the grains of silt — or the pieces of a substance such as gravel — the more quickly they sink, the more frequently they have to be thrown up, the greater is the value of V needed to carry them at all, and the less is the full charge of them for any given values of V and D. In mixed silt — say of fine and coarse sand — the full charge is somewhere between the full charges of the coarse and the fine. It follows that a muddy stream can carry less sand than a clear stream of similar velocity and depth can carry.

Rolling power depends on V and is probably not affected by D For a given value of V, and any particular kind of

silt, there is no doubt a certain limiting depth, below the bed, down to which movement takes place. There is thus a limiting discharge of rolled material. When the silt is muddy the portion nearest the bed has been found, on the Nile, to consist of "slurry" or semi-fluid mud [1]. This may be considered as rolled material.

It is not yet proved that the velocity of a stream is appreciably affected when it is charged with suspended silt.

It is unlikely that the rolling power of a stream is appreciably affected by its being charged with suspended silt.

If the bed of a stream is hard and if it can carry all the material brought into it, none may be rolled If the bed is soft or of loose material, some is likely to be rolled whether the stream carries any silt or not. Thus there is no possibility of fixing any general proportion between carried and rolled silt In a deep muddy river however, the rolled material is generally much less than that carried.

Generally the carrying power of a stream is alone dealt with in discussions because of the difficulty in measuring rolled material. The latter may however be the chief cause of any silting trouble.

Since both powers depend on V they nearly always increase or decrease together. An exception may occur where there is a great change of cross-section. If, for instance, in the lower reach of a stream, the width is greatly reduced and D greatly increased, while V is slightly increased, the carrying power may be reduced while the rolling power is slightly increased

When, in the case of a full charge of mixed silt, V is reduced, the coarsest kind of silt first drops down to the bed, then the next coarsest and so on. This may also occur even if the charge is not a full one, because V may be only just sufficient to enable some of the coarser materials in the charge to be carried. In either case the silt which drops down may be rolled. When the limit of rolling is reached, deposit of silt on the bed begins.

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[1] *Proc Inst C E*, Vol CCXVI p 183 (Buckley)

Scour of the bed may be effected by rolling or by wearing away. In either case it depends on V.

If a stream has power to scour any particular material from its channel it has power to transport it, but the converse is not always true. If a stream is not fully charged, it tends to become so by scouring its channel, but, except when the material is soft or loose, it has more difficulty in eroding than in transporting it. Earth — especially clay — gravel, shingle or boulders may form a very hard channel.

Let a stream be carrying and rolling sand — all of one degree of coarseness — up to its full capacity and let the bed be of equally coarse sand. The stream cannot scour. If any local or temporary scour occurred the resulting charge would exceed the full charge so that some silt would drop down and nullify the scour. But if the stream enters a reach where the bed is of fine sand it can take up some of it, depositing a smaller quantity of coarse sand. With this new charge the stream can take up some mud and deposit a smaller quantity of its sand.

The silt in a stream is, at first, what is brought into it. It may be able to transport it all. It may have to deposit some kinds though able to carry more of other kinds. It may or may not be able to obtain silt from its channel.

Silting and scour are regular or irregular according as the channel is regular or irregular. In a fairly uniform stream the "silt wedge" — the depth of silt greatest at the off-take and decreasing regularly till it becomes zero — is well known. Below the off-take of a canal, or at the upstream end of a reach of it, there may be a scour wedge analogous to the silt wedge.

The "critical velocity", V_0 which enables a stream to transport all its silt without deposit and without scour — the bed being supposed to be soft or of loose material — was found by Kennedy[1] by observation of reaches in which the streams were fully charged because they had adjusted their beds by silting or scouring and had attained approxi-

[1] *Proc. Inst C. E*, Vol CXIX p 281

mate average stability, neither silting nor scouring when the whole year round was considered.

The streams were charged with the mud mixed with fine sand, found in the Punjab rivers near the hills from which they issue. The most probable values ot V_0 for such silt are given by the equation

$$V_0 = .95\,D^{.57} \quad . \; . \; . \; . \; . \; . \; (1)$$

and are approximately as follows:

$D =$	1	2	3	4	5	6	7	8	9	10
$V_0 =$	.95	1.41	1.77	2.10	2.38	2.67	2.90	3.11	3.33	3.54

Kennedy's formula was

$$V_0 = .84\,D^{.64} \quad . \; . \; . \; . \; . \; . \; (2)$$

and this gave velocities slightly less than the above for the smaller depths and very slightly greater for the greater depths.

The above values of V_0 corresponded to a charge of silt in which the sand was about $\frac{1}{3,300}$ of the volume of water, the charge of mud being probably greater [1]. The sand was of various grades from the finest to coarse and the quantity which was coarser than the $\overline{.10}$ class was about $\frac{1}{35,000}$ of the volume of water. The observations were made at Garhi, 26 miles from the head of the Sirhind Canal.

When the sand, instead of being of all grades was entirely of coarse grades the fraction $\frac{1}{3300}$ became $\frac{1}{9000}$.[2]

For a case in which the volume of rolled material was ascertained see Chap. IV., Art. 2.

For reaches or branches of Punjab canals distant from the heads, or for head reaches taking off lower down the rivers, the sand is finer and the above values of V_0 should be multiplied by $\frac{3}{4}$ to $\frac{9}{10}$. In other provinces and countries the sand is also finer or coarser, and the following multipliers

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[1] *Punjab Irrigation Paper No* 9 Silt and Scour in the Sirhind Canal 1904

[2] These two fractions are compared by Kennedy The degree of coarseness of the "coarse grades" is not stated The figures given in Chap IV, Art 1, show that when the total charge was $\frac{1}{3,000}$ that of sand coarser than the $\overline{.10}$ class was only $\frac{1}{35,000}$ The figures throughout appear to refer to charges of sand and to exclude mud They also exclude all rolled material.

can be used, Egypt $\frac{2}{3}$, Sind $\frac{3}{4}$, Madras (Cauvery and Kistna rivers) $1\frac{1}{5}$. These multipliers have been found to be approximately suitable. It is not certain that it is only the degree of fineness of the sand that is variable. In cases where the multiplier is less than 1 the proportion of the mud to the sand may be greater. In Burma the multiplier $\frac{19}{20}$ is considered suitable. In the case of the Alamo Canal supplied from the Colorado River the multiplier has been found to be $1\frac{1}{6}$. The silt appears to have been mostly mud but the charge heavy [1]. In the rivers of the British Isles and many other countries, the values of V_0 — even in floods — are probably less than in any of the above cases and far less again in ordinary flow apart from floods.

Kennedy's values of V_0, differ slightly — as already stated — from those given above. These velocities have been quoted by one writer simply as being suited to streams of the corresponding depths. If — as is likely — the water is far from being so much charged as was the case in Kennedy's channels, and if the bed is soft, such velocities will cause scour.

For a statement of velocities suited to different soils see Chap. VI., Art. 6.

The old idea was that silt-transporting power depended only on V. In the case of irrigation distributaries the cross sections were somewhat narrow and deep. Silt deposited in the head reaches and was repeatedly cleared out at great expense. A higher bed level at the off-take, a reduced value of D, a steeper slope and a wider channel would give no higher velocity than before and therefore no-one tried it. When Kennedy showed that reduction in D would remedy the trouble, the plan was tried and has since been adopted all over India with conspicuous success and elimination of expenditure.

Kennedy's observations were not sufficiently numerous and precise to give exactitude to his formula. The true index of D may differ from .57 or from. 64. But this is of minor consequence. In designing a channel in which a stream fully charged with Punjab river silt is to flow, the approxi-

[1] *Proc. Inst. C. E.*, Vol. CCXVI. p 172.

mate relation of V to D can be found, or at least one quantity or the other can be kept in the ascendant according as silting or scour is most to be guarded against.

Since a stream can roll or carry any solid body more easily the less its size, the coarser materials are deposited first and picked up last. The stream is constantly sorting out the materials on its bed. Large boulders exist only down to a certain point, smaller boulders, shingle, gravel, coarse sand and fine sand following in succession.

Most streams vary greatly at different times both in volume and velocity and in the quantity of material brought into them. Hence the action is not constant. A stream may silt at one season and scour at another, maintaining a steady average. When this happens, or when the stream never silts or scours appreciably, it is said to be in "permanent regime" or "stable". It may be stable except in flood time. Most natural streams in earthern channels are either just stable and no more, or are unstable.

When a channel is sandy the longitudinal section is often a succesion of small abrupt falls. After each fall there is a long gentle upward slope till the next fall is reached. The sand is rolled up the long slope and falls over the steep one. A fall does not extend straight across the bed but zigzags, so that the channel as viewed from above has a wave-like appearance.

Floods. — In a steady flood the stream has probably an increased carrying power and certainly an increased rolling power. In a rising flood V is greater, and in a falling flood less, than in steady flow[1]. The tendency is for the rising flood to cause scour and for the falling flood to cause silting. On the Irrawaddy it was found that on the day of a high flood, a great deepening of the channel occurred at all the observation sites[2]. This may have been due either to the rise or to the greater depth of water after the rise, or to both. At one site the bed scoured 36 feet. It silted up again in about a month. When water heavily charged with silt

[1] *Hydraulics*, Chap IX Art 3

[2] *Note on the Irrawaddy River*, Samuelson (Government Press, Rangoon).

enters a stream high up during a flood some of it may deposit quickly in a lower reach, the rest as the flood falls. When the upper part of the stream brings down clear water to the lower part it may again pick up the silt and become more or less turbid.

When there is a regular flood season, the floods which bring most silt into the streams are the earliest ones. They sweep in the dust and debris — including earth disintegrated by frost — which have accumulated during the previous half-year or longer period.

If a stream has a considerable discharge for a fairly long period each year, exclusive of floods, it is likely to keep its channel clear; but if not, its capacity is likely to deteriorate owing to deposits.

Observations on the Tigris at Amara by Lewis [1] show that during the six months — June to November — in which the stream was at first gradually falling and was then steady, the mean monthly percentage of silt in the water, by weight, varied from .06 to .012, and was almost exactly equal to $.008\ V^2$, where V is the mean monthly velocity of the stream. V varied from 2.73 to 1.31 feet per second. In the other 6 months there were floods, and the silt percentage was generally much higher than the above. From January to May the heights of the floods increased (Fig. 97, Chap. VII., Art. 1) and so did V, but the silt percentage diminished. The probable cause of this has been mentioned above. The silt was always mud. In a mixture of the silt and water, in which the silt was about 13 per cent., by weight, the silt settled and contracted for 24 hours and then practically ceased to contract. Its volume was then five times what it occupied when dry. It was concluded that "it must be almost liquid in nature and will settle only in very still places".

The maximum charge of silt in the first six months was in June and was .06 per cent. by weight or $\frac{1}{1667}$. The ratio to the water by volume would be about $\frac{1}{2880}$ but it is highly improbable that the stream was fully charged. From July to November inclusive, V and D were less and the charge

[1] *Proc. Inst. C E*, Vol. CCXII. p. 393.

was less. In January the charge was nearly 7 times that in June, or $\frac{1}{422}$ by volume. It is not stated that any deposits occurred in the river channel below Amara but that "it must be concluded that most of this fine silt is carried into marshes or right out to sea".

Weir or Abruptly Raised Bed. — The effect of a weir is very variable. If the velocity just above the weir is not less than V_0, the stream can transport all the silt until just above the weir. The silt which is carried of course goes over. Rolled material, such as shingle or gravel, is extremely likely to be deposited. Sand is much less likely to be deposited. The velocity of the stream may be such that the eddies above the weir stir up the rolled material and carry it over. The above remarks apply also to a sudden rise in the bed of a channel and to the case of an off-take where the bed of the branch is higher than that of the main channel. In the case of a weir there may be a mere local deposit, the material forming a slope up towards the crest so as to bury part of the upstream face of the weir thus reducing the height to which the material has to be lifted. If a general deposit occurs in the upstream reach, it may extend for a great distance, as in the cases of weirs on some Indian rivers where the bed is of boulders and shingle or even of sand. The promiscuous building of weirs, groynes and other works on small rivers is a fertile cause of their "deterioration" by silting.

In the case of deposit upstream of a weir it has been stated [1] that the deposit begins, not at the weir but at the furthest point to which the afflux extends. This may of course happen but the statement is far from being generally correct. The quantity of silt in the stream may be such that deposit does not begin till near to the weir. The case referred to was probably that of a weir in a channel not very far from its off-take. In this case the silting would be partly due to the weir and partly an ordinary silt wedge.

Art. 3. Action on Banks and at Bends. The laws stated in the preceding article probably hold good in the case of any portion of a stream flowing over a gently shelving

[1] *Indian Engineering*, 21st. Sept. 1918.

bank. Silt however, tends to deposit in the angle where the slope meets the water surface, and the bank to become very steep or vertical. Such banks are extremely common.

When the bank is steep or vertical, questions of silting and scour of the bank are not complicated — as in the case of the bed — by the presence of rolled material or by suspended matter dropping down. Deposits on the bank depend on V and on the charge of silt. Scour of the bank appears to depend only on V, the power of the stream to wear away the bank being probably very much the same whether or not it is charged with silt. The velocity very near to a steep bank is low, relatively to that in the rest of the stream. Thus there is often a tendency for silt — generally mud not sand — to deposit on the bank, and for the side to become vertical except for a slight rounding at the lower angle. A bank may receive deposits when the bed is receiving none, and it may have a persistent tendency to grow out towards the stream. The growth of the bank is sometimes regular,its line being well preserved, but it may be irregular, especially if there were irregularities at first either in bank or bed, causing eddies, or if clumps of vegetation become established on the new deposits. The question of growth or scour of banks is intimately connected with that of vegetation. There may be scour of the bed and silting at the banks or vice versa. The material of the bank may be quite different from that of the bed.

When scour of the sides of a channel occurs, it may be by general action of the stream on the sides, or by action at or near the toe of the slope — especially if scour of the bed has taken place — which causes the upper part of the bank to fall in. Falling in is generally irregular, and the bank presents an uneven appearance. The fallen pieces of bank may remain, more or less intact, especially if they are held together by the roots of grasses, where they fell, and may — temporarily at least — prevent further scour occurring along the toe of the slope. Falling in of banks is most liable to occur in large streams and with light soils. It depends largely on the nature of the soil and on the extent to which it is

protected by vegetation and the roots of trees. It may be caused by the waves which are produced by steamers and boats or, especially in broad streams, by wind. In slow streams it is extremely likely to cause silting of the bed.

The lower a bank is, the less is the quantity of material which the stream has to deal with and the more easily it can be eroded. This is probably one reason why erosion is so much less on the Mississippi, where the height of the banks from ordinary water-level to flood level is often 40 feet, than on the Indus where it is about 10 or 12 feet. Erosion, or "caving" of the banks of the Mississippi is greatest where the ground level is low.

In channels in alluvial soils the falling in of banks sometimes occurs more when the stream is falling than at other times. This has been noticed on both the Mississippi and the Indus, but not so much on the latter. The causes are the removal of the water pressure, the wet state of the soil and to some extent the removal of sand or soil from the interior of the bank by the water which drains out.

At a bend, owing to the action of centrifugal force and to tranverse currents caused thereby, there is a deposit near the convex bank and a corresponding deepening — unless the bed is too hard to be scoured — near the concave bank. The water-level at the concave bank is slightly higher than at the convex bank.

As the transverse current and transverse surface slope cannot commence or end abruptly, there is a certain length in which they vary. In this length the radius of curvature of the bend and the form of the cross-section also tend to vary. It can often be seen in plans of river bends, that the curvature is less sharp towards the ends. The abnormal section — deepening at one side and shoaling at the other — may not begin till some way downstream of the commencement of the bend but it generally continues to the extreme end.

When once a stream has assumed a curved form, the tendency is for the bend to increase. The greater velocity and greater depth near the concave bank react on each other

each inducing the other. The concave bank is worn away, or becoming vertical by erosion near the bed, cracks, falls in, and is washed away, a deposit of silt occurring at the convex bank, so that the width of the stream remains very much as it was. The bend may go on increasing and it tends to move downstream.

Comparing the sectional area of the stream at a bend and in the straight reaches near it, there is generally no great difference and it cannot be said that either tends to be in excess, at least in cases where the stream can scour near the concave bank. Where — owing to hardness of the channel or to low velocity — it cannot do so, the deposit near the convex bank may reduce the sectional area. As regards the width of the stream, the convex bank at a bend may — owing to the silt deposit — have a somewhat flat slope,

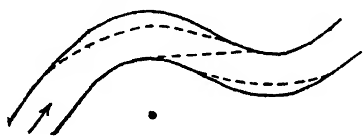


Fig 5.

while in the straight reaches both banks are steep. Hence, at a bend at low water — and in the ordinary stages of the stream — the width of the stream is often less than in the straight reaches while the depth is greater.

In Fig. 5 the deep places are shown by dotted lines. Along the straight dotted line there is no deep place. Such a line would be used for a ford. At low water it becomes a shoal. This is the chief reason why a tortuous stream at low water consists of alternate pools and rapids. It is sometimes said that deep water occurs near to a steep hard bank. Such deepening is local and due to bends or to obstructions which give the current a set towards the bank, or it is due to eddies caused by irregularities in the bank or channel. In a straight channel with even and regular banks there is no such deepening.

Sometimes bends, especially if changes are in progress,

are irregular or contain angles. In such cases, and also where short bends closely succeed one another, it may be difficult to say how they affect the depth and cross-section of the stream.

Art. 4. General Tendencies of Streams. Since the velocity is greater as the area of the cross-section is less, a stream always tends to scour where narrow or shallow, and to silt where wide or deep. The cross-section thus tends to become uniform in size. Suppose two cross-sections to be equal in size but different in shape. The velocities in the two sections are equal. The tendency of the bed to silt is greater at the deeper section and, when silting has occurred on the bed, the section is reduced and there is a tendency to scour at the sides. Thus the cross-sections tend to become also uniform in shape.

If there is a submerged silt-bank extending over a considerable length of stream, the stream tends to scour it away and in doing so to deposit silt below its downstream end. Thus such a silt-bank often moves down a stream — this can be seen on a small scale in roadside channels — and it probably produces a gradual reduction in the water-level at the original position of its upper end.

Owing to the tendency to scour alongside of, or downstream of, obstructions, it is clear that a stream constantly tends to destroy them.

When a bend has formed in a channel previously straight, the stream at the lower end of the bend, by setting against the opposite bank, tends to cause another bend of the opposite kind to the first. Thus the tendency is for the stream to become tortuous and, while the tortuosity is slight, the length, and therefore the slope and velocity, are little affected; but in cases in which the general velocity of the stream is high or the banks soft, the action may continue until the increase in the length of the stream materially flattens the slope, and the consequent reduction in velocity causes erosion to cease. Such a stream during a flood may find, along the chord of a bend, a direct route with, of course, a steeper slope. Scouring a channel along this route it straightens itself,

and its action then commences afresh. Short cuts of this kind — also called "avulsions" — do not, however, occur anything like so frequently as is sometimes believed. In some streams of particular classes the bends become loops (Art. 5 and Fig. 10) that is the length of the bend is very much greater than that of the chord across the neck of land. There is a great difference between the water levels at the two ends of the chord, and a steep slope along the chord and short cuts may then occur. Otherwise they are extremely rare. V increases only as $\sqrt{S}$, and if the country is covered with vegetation it is most difficult for a stream to scour out a new channel.

In a channel there is sometimes a "bar" or local deposit of silt which extends across the bed. It exists in a wide place and merely makes the sectional area of the stream the same as elsewhere. Any other silt bar would form a kind of submerged weir and the velocity over it would give rise to scour which would destroy it. A bar of hard material can of course exist anywhere. The sand bar at the mouth of a river does not form in the river itself unless it has widened out. It is sometimes suggested that a silt bar can exist in a place which is not wide, but the term "bar" is then probably used for deposits which are not strictly local, for instance the silt wedge described above (Art. 2).

There is an obvious tendency for silt to deposit where the bed slope of a stream flattens and for scour to take place where it steepens, and thus the tendency is for the slope to become uniform.

In a natural stream flowing from hilly country to a lake or sea, the slope is steepest at the commencement and gradually flattens. There is thus a general tendency for the bed to rise owing to silt deposit. This rising tends to increase the slope and velocity in the lower reaches, and this again enhances the tendency of the stream to increase in tortuosity.

Of all the material composing the bed of such a river, the mud and fine sand alone are carried into the sea. The reaches of coarse sand, gravel, shingle and boulders keep advancing slowly and probably becoming slowly raised.

In the case of a river flowing through flat country the soil is generally alluvial and soft; but hard material may occasionally occur in the banks or bed. Any such conditions may of course greatly affect the action of the stream.

The action of the stream in increasing its tortuosity and length takes place, as above stated, because its velocity is too great for its channel. In the absence of short cuts the stream in time attains an average stability, that is its regime is upset only because of the fluctuations in its discharge. Any short cuts which occur, allow the floods to get away more readily but are no part of a scheme for establishing stability. They start a fresh set of changes and are instances — akin to storms and tidal waves — of the restless play of natural forces.

When a silt-bearing stream overflows its bank the depth of water on the flooded bank is generally small and its velocity low — particularly if the bank is covered with vegetation or obstructions — and a deposit of silt takes place on the bank. The water of subsequent floods flows over it and, deposits its silt further away from the stream. In this way a strip of country along the stream gradually becomes raised, the raising being greatest close to the stream. In other words, the stream runs on a ridge. If the bank becomes raised so high that flooding no longer occurs, the raising action ceases, but if, as is likely in alluvial country, the bed of the stream also rises, the action may continue and the ridge become pronounced. The level of the river becomes raised and the liability to severe floods is increased. It is however pointed out in the paper — mentioned above — by Lewis, that since the silt-laden water brought into a river by a flood may not arrive in the lower reaches till the peak of the flood has passed, there may be little silt deposit on the banks in those reaches.

Any lake or swamp occurring in the course of a river acts as a reservoir and mitigates the severity of the floods lower down. The river channel itself acts as a reservoir, Also the flood waves flatten ont. The greater the distance from the source the less are the relative fluctuations in the discharge.

The mean water-level at the mouth of a river is the mean sea level. The mean surface slope of the river near the sea is steepened in floods and flattened during low supplies.

Owing to the raising of the bed of a river an avulsion can occur even when there is no bend to be cut off. The most notable case is that of the river Hoang Ho or Yellow River, also known as "China's Sorrow". The basin contains much loess which is washed into the stream and renders it highly silt-laden. Where the river passes through the plains of China, the bed is raised by silt deposit and great lengths of embankment are made to prevent floods. The bed rises further and the embankments are raised, Now and then a breach occurs and, if it cannot be repaired, the river leaves its old channel and finds a new course to the sea, generally with prodigious loss of life and property. In 1851 a breach occurred near Kaifungfu in Honan, where the bed of the river was some 25 feet above the ground, and the stream took a new course along the channel of a small river — the Tsing — to the sea. In 1887 a breach occurred which took more than a year to close, was accompanied by appalling loss of life and flooded an area of many thousands of square miles.

Avulsions may be caused by geological changes. A great change in the course of the Brahmaputra has been attributed by Gales to a gradual upheaval of the land in the Madhupur district of Bengal [1].

In the case of the large Bhimbar torrent which it was proposed to carry under the Upper Jhelum Canal, it is mentioned by Sir John Benton [2] that the torrent has raised its bed and runs on a ridge and that it might change its course. The chance of this occurring may, in this particular case, have been appreciable but in most rivers the chances of avulsions of this kind are as remote as of those due to bends. See also Art. 5.

It is common for flood water from a river to spill over the river valley — and perhaps to travel down definite channels — soon after the river has left the hills, and for the spills to

[1] *Proc Inst. C E.*, Vol. CCV p 22
[2] *Proc Inst C. E.*, Vol CCI. p. 59.

continue for a long distance so that, during floods, the discharge passing down the river channel as it approaches its mouth, becomes less. In such a case the capacity of the channel — generally the width — becomes less. The spill channels may be wide and shallow. The spill water, spread over a large area and its flow being intermittent, is subjected to rapid losses from absorption, percolation and evaporation. At Baghdad the Tigris is 1,150 feet wide and discharges, in floods, about 194,000 c.ft. per second. At Kurna 253 miles lower down, the width is 590 feet and the discharge is about 44,000 c.ft. per second.

Bifurcations and Junctions. — Natural bifurcations of streams are most common in river deltas and in shifting rivers or where there is flooding. The angle of bifurcation, A B C (Fig. 6) is generally more than 90°. The corners D and E

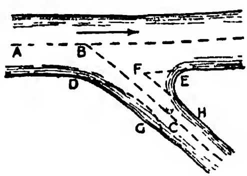


Fig. 6.

become rounded and this gives the branch some advantage as regards loss of head. There is usually a silt bank at DG because the bulk of the water entering the branch is flowing round a bend. Thus the actual off-take often tends to be at about 90°.

A branch may be an artificial channel or it may come into existence as a branch if the river shifts its channel and cuts into it. The Sutlej cut into the Beas but it did so scores of miles above the original junction and thus caused, not an avulsion but an abandonment by the Beas of a great length of its channel.

As regards the supposed tendency of a branch to draw the main stream towards it, when a branch comes into existence it, of course, causes a draw-down in the main

stream. If the latter is wide, the fall in the water-level at D (Fig. 6) is greater than at the opposite bank. The stream has thus some tendency to scour the bank at D and upstream of it. Below the bifurcation there is some tendency for silt to deposit (see also Chap. IV., Art. 1) but this also applies to the branch.

A branch can cause an avulsion only when the conditions are such that it can scour and enlarge itself. It is just as likely to deteriorate by silting and finally to disappear.

Near Dera Ghazi Khan, where the Indus habitually erodes its western bank, there was a proposal to enlarge an inundation canal which had its off-take on the western bank. The discharge of the enlarged canal would not have been

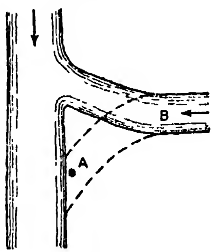


FIG. 7.

three per cent. of that of the river in its ordinary stages and one per cent. in floods. The slope of the canal was flat. It would have silted and could never have scoured. Nevertheless strong opposition was raised, to the proposed enlargement, by persons who were not professional engineers. They merely knew that draw could be dangerous under some circumstances.

At the junction of one stream with another the tributary stream, if steep and rapid, may bring down shingle or gravel which may deposit in the main stream. When the main stream is not much larger than the tributary, the latter may cause a set of the current against the opposite bank and erode it. When the tributary stream enters the main stream so as to partially oppose it (Fig. 7) the eddying and disturbance

may be considerable — in the case of floods great — and the effect may be very much the same as if the main stream itself had an elbow of 90°. A correction can be effected by an alteration of the tributary in the manner shown by the dotted lines.

Art. 5. Rivers. There are various types of river. Some rivers have very wide and soft channels which are only filled from bank to bank in floods, if then. The characteristics of this type are exhibited in an extreme degree in the rivers of the Punjab . These vary in width from ¼ mile to 4 miles. There is nearly always one main stream and several minor channels or "creeks" though these may be dry in the lower stages of the stream which is from November to March inclusive. The channels wind about among sand-banks of great extent.

The streams, especially the main stream, constantly shift their courses by scouring one bank or theother. The Indus sometimes cuts into its regular bank 100 feet or more in a day, and it may cut for half a mile or more without cessation, trees and buildings disappearing into the stream. Now and then the main stream takes a short cut, either down a minor arm or across an easily eroded sandbank. This is a totally different matter from a short cut across high ground. The great changes occur in the floods. The changes are wholly irregular and generally impossible to forecast. The sandbanks, until they are cut away, keep receiving fresh deposits of silt during high stages of the river and the upper layers are often of good soil. They speedily become covered with tamarisk or — especially when they adjoin the banks of the river — are brought under cultivation.

The melting snows of the Himalayas cause the streams to rise in April. They attain their greatest height in July and August and fall in September and October. In June, July, August and September there are also heavy rains in and near the hills, and much fluctuation of the water-level. The highest flood of the year is generally 8 to 12 feet above the winter or low water level. The bank level is generally within 2 or 3 feet of the high flood level. It may be above it

or below it. The slopes of the rivers at places far from the hills average about a foot per mile. Nearer the hills they are steeper. The velocity of the main stream in winter is 3 or 4 feet per second but in floods it may be 6 or 8 feet per second.

The depth of water during the low water stage may be 10 or 15 feet. In floods it may attain 50 feet in places. The material scoured away is partly carried along by the stream and partly deposited on the sandbanks.

The water level depends partly on the course of the stream. Suppose the surface gradient to be a foot per mile and the course to alter from that shown by the firm lines in Fig. 8, to that shown by the dotted line, the water level at *b* will

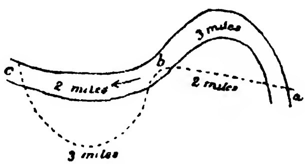


FIG. 8

be a foot higher than before, although the general level of the stream *ac* is supposed to be unaltered.

Again, the water level in a creek is generally different from that in the main stream opposite to it. If the water levels at two points, one in the river and one in a creek, are equal, to begin with, the silting up of the head of the creek, which usually occurs, will soon cause the water level in the creek to be lower than that in the main stream. A difference of two feet is nothing uncommon. This, in a great measure, explains why a portion of land which is flooded one year may escape flooding in another year, although there may be just as much water in the river as before. When the main stream is near to, say, the west bank, the floods on the west bank are likely to be most severe.

The tortuosity of such a stream increases as it gets nearer the sea, The actual length of the Indus in the 400 miles nearest the sea is 39 per cent. greater than its course measur-

ed along the bank, In the reach from the 600th to the 700th mile from the sea, the difference is only 3 per cent.

The enormous variations — both as to time and place — in the rainfall and in the quantity of melted snow and the consequent endless variations in the quantities, times, places and characters of the silt brought into the rivers, and in their discharges, amply suffice to account for the changes in the channels and for the impossibility of predicting them. The heavy charges of silt cause deposits in the channels,

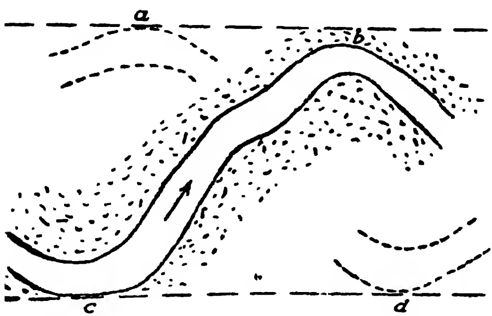


FIG. 9.

especially when a flood is subsiding. The next rising flood — or a period of clear water — moves the silt on. It may also scour the channel. Thus there are constant changes in the surface slopes, depths, widths and velocities.

Some kinds of change, however, only occur — as a rule — within certain limits. Avulsions occur every day, within the river channel but are practically unknown outside it. As regards bank erosion, since the river can erode its bank more easily the less the height of the bank (Art. 3) it can obviously erode a low sand-bank in its channel more easily than either of its regular banks. Great as are its inroads into land outside its channel, they are far less than its inroads into another part of its channel. If the points *a, b, c, d* (Fig. 9) are the extreme limits which the stream has been known to reach, it is improbable that it will — in the absence of

any special causes — move far outside the boundaries shown by the dotted lines.

The above description refers, as stated, to the rivers which flow through the Punjab plains. The main characteristics are great width — in proportion to the depth of water — frequent splitting up of the stream and great and frequent changes. These characteristics also exist in the sub-montane reaches of the Punjab rivers and, with the characteristics less marked, in other rivers of India — for instance the Godavari and Kistna — and in rivers in some other countries. The Mississippi and some of its great tributaries, as well as very many other well-known rivers, are of a different type. They are not so wide, much deeper, far less split up into islands and channels and less liable to shift their courses. The Mississippi discharges far more water than the Indus but the width of its channel is not a third as great. Its mean depth from bank to bank is several times as great.

The causes of the existence of these divergent types of rivers seem to be as follows. The streams of the Mississippi type are not highly silt-laden and therefore a moderate velocity enables them to maintain a great depth of water. The velocity of the stream, as its bed rises, becomes too great for its channel, the bends become loops and short-cuts (Fig. 10) occur [1]. The slopes of the valleys are flat. If the stream merely brought down little silt from the hills but had then a considerable velocity it would obtain silt from its channel and might become of the Indus type.

In rivers of the Indus type the streams are highly charged with silt. They cannot therefore maintain a great depth of water. They can, however, scour away their banks. Instead of becoming deep they widen their channels. As in the case of all other rivers, the stream works into bends but the stream is, at most times, very far from filling its channel and it forms the bends within its channel. These bends fail to become loops because of the ease with which

[1] On the Mississippi there is on the average one "cut-off lake" — a name given to the crescent-shaped lakes which are formed from the abandoned loops — to every 17 miles of channel.

the stream can straighten itself during floods by merely cutting across low sand-banks or down creeks. The slopes of the valleys are comparatively steep. If they were flat the streams would, by overflow and deposit, have reduced their charges of silt and would be of the Mississippi type.

The Ganges and Brahmaputra are of an intermediate class. There are of course gradations and many other intermediate types. British rivers are generally of the narrow and deep class.

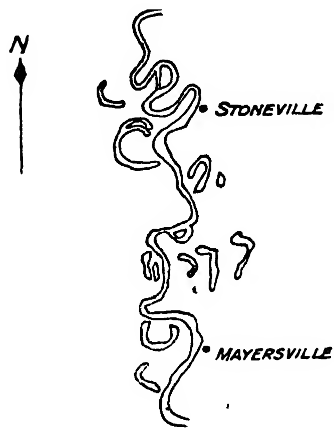


Fig. 10

The Jhelum where it traverses the Vale of Cashmere is also of the narrow and deep type. After passing through all the usual stages and becoming navigable for a length of 60 miles — in the course of which it passes through the Woollar Lake — five thousand feet above sea level, it starts afresh on a torrential career, descends some three thousand feet and emerges into the plains of the Punjab.

All rivers which, like those of the Punjab, are chiefly fed from melting snow, exhibit a similar general regularity of rise and fall, but superposed on this are enormous irregularities, in the flood season, due to rainfall.

On the Mississippi at Vicksburg some 324 miles from the

sea, the surface fall per thousand feet has been found to be from .022 to .064 feet; at Carrollton, nearer the sea, falls of .020 and .017 feet were observed, some much flatter slopes also occurred but were too small for accurate measurement. The maximum flood discharge is about 1,900,000 c.ft. per second. The slope of the Nile in Egypt is about .09 feet per thousand and the flood discharge about 300,000 c.ft. per second. The slope of the Meuse is .475 feet per thousand feet and the flood discharge 70,000 c.ft. per second.

There is no approach to any fixed proportion between the flood discharge of a river and the low water discharge. The latter may be negligible or non-existent.

CHAPTER IV

THE CONTROL OF SILTING AND SCOURING ACTION

Art. 1. General. Most important works which affect the regime of a stream have some effect on its silting or scouring action. Such works are dealt with in other chapters and the effects which they are likely to produce are considered. The present chapter deals with only those measures whose chief object is to cause a stream to alter such action. The object may be direct, that is, concerned only with the particular place where the effect is to be produced, or indirect, as for instance where a stream is made to scour in order that it may deposit material further down the stream. But any alteration of silting or scouring action must produce some effect at places lower down the stream and this should always be considered.

Production of Scour or Reduction of Silting.—Sometimes the silt on the bed of a stream is artificially stirred up by simple measures, as for instance, by scrapers or harrows attached to boats which are allowed to drift with the stream, or are propelled downstream, or by means of a cylinder which has claw-like teeth projecting from its circumference and is rolled along the bed, or by fitting up boats with shutters which are let down close to the bed and so cause a rush of water under them, or by anchoring a steamer and working its screw propeller. It is thus possible to cause a great deal of local scour, but the silt generally tends to deposit again quickly, and it is not easy to keep any considerable length of channel permanently scoured. The system is suitable in a case in which a local shallow or sandbank is to be got rid of and deposit of silt a little further down is not objectionable. It may be suitable in a case in which the bed is to

be scoured while a deposit of silt at the sides of the channel is required, especially if some arrangement (see below, also Art. 3) to encourage silt deposit at the sides is used.

Holding back the water by means, for instance, of a regulator or set of sluices, and letting it in again with a rush may, if frequently repeated, have considerable effect in moving silt on in the reach immediately downstream. Regarding the upstream reach, it has been remarked (Chap. III. Art. 2) that a weir does not necessarily cause silt deposit. If, in a stream which does not ordinarily silt, sluices cause some silt deposit when the water is headed up, the cessation of the heading up not only removes the tendency to silt, but the section of the stream, at the place where the deposit has occurred, is less than before and thus there is probably a tendency to scour there. If a regulator is alternately closed and opened — especially at short intervals — a permanent deposit of much consequence is hardly likely to occur upstream of it. The device, of course, does not apply to a case — the head reach of an inundation canal for instance — in which the stream had a tendency to silt before it was headed up.

A stream may be made to scour its channel by opening an escape or branch, This causes a draw-down in the stream, and an increase in velocity for a long distance upstream of the bifurcation. This procedure is sometimes adopted on irrigation canals. The escape is generally opened on emergencies, in order to reduce the quantity of water passing down the canal. It may be opened solely to induce scour or prevent silting, but in this case valuable water runs to waste. The floor of the escape head is usually higher than the bed of the canal, but this does not interfere with the scouring operations except at times of low supply. It causes less rolled material to be passed out of the escape and more to go down the canal.

If there is a weir in the river below the off-take of the canal, and if the escape runs back to the river and thus has a good slope and a heavy discharge, the scouring action in the canal may be very powerful.

If the canal has a uniform slope and cross-section throughout,

the slope of its water surface is — when the escape is open — greater upstream of the escape than downstream of it and there is thus an abrupt reduction of velocity and possibly a deposit of silt in the main channel below the escape. This may or may not be objectionable . In the case of an rirrigation canal, it is far less objectionable than deposit in the head of the canal. The best point for the off-take of any escape or scouring channel depends on the position of the deposits in the main channel. The off-take should be downstream of the chief deposits, but as near to all of them as possible. A breach in a bank acts of course in the same way as an escape.

A stream of clear water when sent down a channel will scour it if the material is sufficiently soft. In the case of the Sirhind Canal[1] (Chap. III Art. 2) when the river water became clear after the floods, the proportion of coarse sand, that is, sand above the $\overline{.10}$ class, carried by the canal water was about $\frac{1}{15,000}$ by volume. This was in the period from 22nd September to 7th October. From 8th to 23rd October the proportion averaged $\frac{1}{32,000}$, from 24th October to 8th November $\frac{1}{44,000}$, and from 9th to 24th November $\frac{1}{85,000}$. The reason of this reduction was that the comparatively clear water kept picking up the sand from the bed and moving it on, the finer kinds being moved most quickly. As the coarse sand left on the bed became less in quantity, the water took up less. It appears, however, that the water also picked up some mud. Also that the total suspended sand late in November was $\frac{1}{9,000}$ of the water. All the observations mentioned in this paragraph were made at Garhi, 26 miles from the head of the canal.

Production of Silt Deposit. — Works or measures for causing silt deposit may be undertaken in order to cause silt deposit in specific places where it will be useful, or in order to free the water from silt. Sometimes both objects are combined.

If a stream can be turned into a large pond, marsh or low

[1] *Punjab Irrigation Paper*, No. 9, Silt and Scour in the Sirhind Canal, 1904.

ground—a bank being built round it if necessary—it can be made to part with some or all of its silt whether rolled or suspended. The silting up of marshes, pools, borrow-pits, [1] etc., is now being effected, or should be effected, in places where mosquitoes and malaria are prevalent. This device is also used when water containing sand is supplied to turbines or other machinery. If the settling tank is not too large the sand can be periodically cleared out.

In the upper or torrential part of a stream, pits may be dug in the bed and cleared periodically, or a high dam, provided with a sluice and a high-level waste weir, may be built across it. The space above the dam becomes more or less filled with gravel, etc. This has been done in Switzerland [2]. In the United States long weirs have been built in order to stop the progress of detritus from gold mines. Such detritus was liable to choke up rivers and damage the adjoining lands. Similar damage has occurred on the river Derwent in Cumberland. The detritus from hill torrents can also be reduced by afforestation of the hill sides (Chap. II. Art. 6).

The "silt-traps" used on irrigation canals in America and other countries, are similar in principle to the pits and weir basins just described. They are used chiefly for trapping sand. Since their periodical clearance by excavation would necessitate closure of the canal or the use of dredges, they are used only where a drainage channel crosses or runs close to the canal, and are scoured out periodically by letting the canal water escape into such channel. Fig 11 shows an arrangement which is suitable where the canal water can conveniently be held up by a weir. The weir is shown by a dotted line. The upper diagram shows the canal in flow and silt being trapped, the lower one the canal water shut off and silt being scoured.

In another kind of trap (Fig. 12) there is a basin in the canal. The gates A, A, can be opened and the silt washed out. It is an improvement if there is a grating, in the position shown by the dotted line — formed of iron bars whose cross-

[1] As to these see Chap. VI, Art. 6.
[2] *Proc. Inst. C. E.*, Vol. CLXXI.

section is a triangle with its apex upwards — through which sand drops. Also see Chap. VI., Art. 1.

The head of a canal at Ventavon in France, widens out into a masoury basin in whose bed are twelve holes and through them gravel and shingle fall and are washed out.

In all the above cases there must be a sufficient fall — from the water level in the canal to that in the drainage or escape channel — to enable scouring to be quickly effected, the canal water must be wasted while scour is going

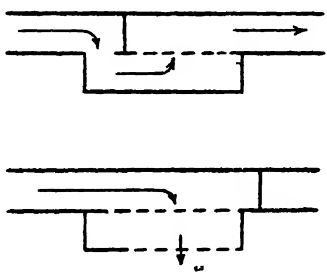


FIG. 11.

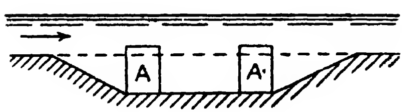


FIG. 12.

on and the drainage or other channel must be such that the scoured material will not choke it. For the above reasons, and owing to the heavy cost, silt traps are not used on very large canals, the largest being one, in the United States, discharging some 800 c.ft. per second.

Yet another arrangement — rarely adopted — is to construct a double channel for the upper reach of a canal, one being used while the other is being cleared of silt.

When a stream is in embankment — irrigation channels are frequently so — the banks can be set back (Fig. 13) and suspended silt will then deposit on the berms, especially when the water on the berms is shallower than shown

on the diagram, or when there is vegetation on them. If necessary branches of trees can be laid on the berm at intervals, and secured to pegs, so as to form spurs; or such spurs can be made of earth. The object of this arrangement is generally to create very strong banks in low ground, while avoiding borrowpits; these are unsightly and — in eastern countries — injurious to health.

A similar plan can be adopted when the berm is only slightly below the water-level and even when it is only occasionally submerged. In this case the deposit of a small bank of silt along the edge of the berm next the stream may prevent the access of fresh supplies of silt-bearing water to the parts further away. Gaps should be cut in the bank

FIG 13

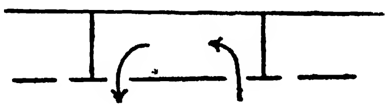


FIG 14

of silt at intervals, and cross banks made to form "silting tanks", as shown (Fig. 14). The inlets to the tank should be large, and the outlets small, so that the water in the tank may have little velocity. It is not, however, correct to have the outlet so small—unless the water contains very little silt—that there is very little flow through the tank. The tanks will generally be silted up most quickly by allowing a good flow through them, even though only a small proportion of the silt in the water is deposited. Regular banks arranged to form tanks on the above principle can be made behind the original banks of a canal in cases where the original banks were not, for any reason, set back, but the arrangement, with its duplicate banks — and cross banks — is expensive. It is also troublesome because of the danger of breaches occurring in the outer bank and because the inlets and outlets need attention.

When a channel is made in low ground and the excavation

is not sufficient to make the banks, borrow-pits are sometimes dug in the bed of the channel. Such pits should not be long and continuous, but wide bars should be left so that a number of short pits will result. These pits will trap rolled material as well as suspended silt. The object in this case is to free the water from silt and, by obliterating the pits, to reduce the size of the channel and thus reduce the loss of water from absorption. In large channels the pits may be dangerous to anyone bathing in, or crossing, the stream. In some soils the bars are liable to be washed out but silt is likely to be deposited ultimately unless the pits extend over a great length of channel.

Protection of the Bed. — It is possible to afford direct protection from scour to the bed of a stream by constructing walls across it. In some streams in Switzerland the wall con-

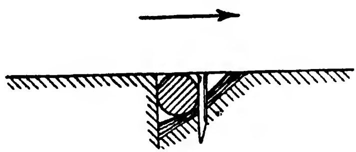


FIG 15.

sists of tree trunks secured by short piles and resting on brushwood (Fig. 15). But as long as the walls are not very close together or raised above the bed they cannot entirely stop scour. If raised above the bed they form a series of weirs. The weirs must be so designed that the depth of water in a reach between two weirs is great enough to reduce the velocity down to the critical velocity, or less. The fall in the water surface at each weir being very small, the discharge over the weir can be found by considering it as an orifice extending up to the downstream water surface, the head being the fall in the surface at the weir.

Sometimes the cross wall slopes steeply up at the ends so as to conform to the intended side-slopes of the channel and form a "profile wall" for both bed and sides. The side portions thus form spurs and if numerous may cause silting at the sides (Art. 3).

To stop scour of the bed by direct protection without raising the water-level, the bed can be paved, a plan adopted in artificial channels with high velocities. The paving can be of stones, bricks, or concrete blocks. The Villa system of protection, which has been used in Italy, France, and Spain, consists of a flexible covering laid on the bed. Prisms of burnt clay or cement are strung on several parellel galvanized iron wires, which are attached to cross-bars so as to form a grid a few feet square. The grids are loosely connected to one another at the corners, and the whole covering adjusts itself to the irregularities of the bed[1]. See also Art. 3 (Fig. 36).

The special protection or paving required in connection with weirs and such-like works is considered in Chap. VIII., Art. 6.

Art. 2. Silting below Off-Takes. Silt frequently gives trouble by depositing in the head reaches of channels taking off from others. The quantity of silt entering a branch channel can be minimised by drawing the supply, as far as possible, from the surface water of the main channel. The bed of the branch should be high and there may be a weir or "sill" in the head. This arrangement may have great effect — increased, of course, if there are shutters or gates which rest on the sill so that the water has to flow over them — in excluding boulders, shingle, or gravel. As regards sand, it has much less effect than might be expected (Chap. III., Art. 2). See also case of Sirhind canal, below.

An extremely common type of bifurcation is that of an irrigation distributary — called in America a "lateral" — from a canal (Fig. 16). Sometimes much trouble is caused by silt deposits. The figure shows the head of an Indian distributary. It has a sill and numerous shallow shutters — generally baulks — which lie one on the top of another. Or there may be a single gate which is lifted and raised by a winch but this does not admit of the supply being taken from the surface water. The wing walls — one of them splayed — correspond with the inner slope of the canal bank.

Some of the currents shown by the arrows, of course tend

[1] *Proc. Inst. C. E.*, Vol. CXLVII.

to raise the sand and carry it over. A sand which sinks only .1 ft., or less, per second is easily carried over. The velocities of the currents depend on the discharge of the branch divided by the sectional area of the approaching stream, FA. In order to reduce the velocities the height NM — from top of gate to upstream water-level — should be a minimum, the necessary discharge being obtained by lengthening the crest of the sill or gate, that is increasing the width of the head opening.

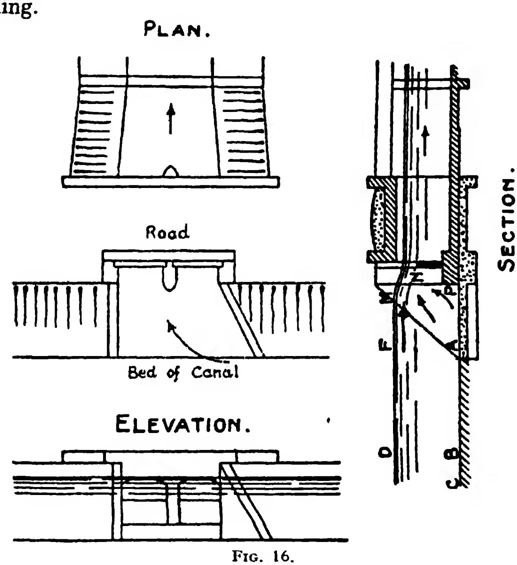


FIG. 16.

There must always be some eddying at an off-take. Fig. 17 shows some different arrangements of the wing walls. On Indian canals, A is used for small off-takes, B and C for larger ones. Probably C tends to minimise eddies and D might possibly be an improvement, but most of the eddies must be caused close to the gate. The upstream side of the sill should perhaps be flush with the gate so that sand cannot lodge on the top of the sill and thence be carried on by another eddy. There should be little leakage through the gate or

around the sides. The tops of the wing walls should be flush with the slope of the canal bank. A plan which has been proposed for adding a curved projecting vane to catch the upper water and divert it into the branch would not remedy the trouble and the vane would be liable to damage.

All that can be done at an off-take like the above is to exclude the silt which is carried — or rolled — by the lower portion of the main stream. The flow downstream of the off-take and of any local eddies, depends on the cross-section, slope and roughness of the branch and on the depth of

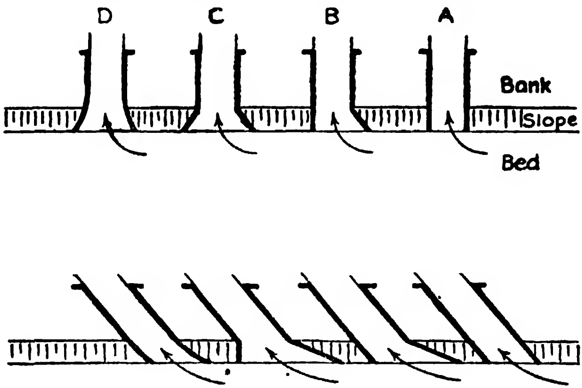


FIG 17.

water in it. The flow is just the same as in any other similar reach remote from the off-take. If this were always remembered there would perhaps be fewer proposals for reducing silt deposit downstream of off-takes.

The question of velocity of approach — apart from questions of silting — is considered in Chap. VI. Art. 1. The lower part of Fig. 17 has reference to it.

An arrangement devised by King[1] consists in fixing a number of parallel curved vanes on the bed of the main channel (Fig. 18) so that the lower water is thrown off and moves away from the off-take of the branch. To replace it,

[1] *Proc. Punjab Engineering Congress*, 1921.

higher water containing less silt is drawn towards the branch, the water in the canal being given a rotatory movement. Upstream of the off-take there is pitching on the bed of the canal for 50 to 100 feet according to the depth of water. Its width is 2 to 3 feet greater than that of the area occupied

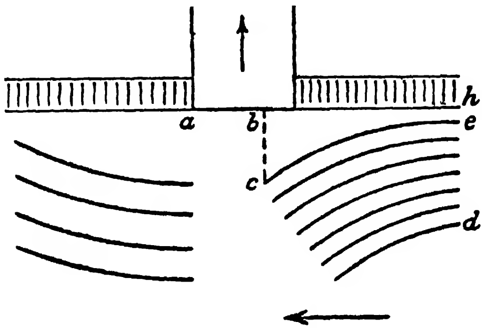


Fig. 18.

by the vanes. Its surface is .5 foot above the canal bed — though it is implied that it need not be so — and this reduces the waterway and tends to cause scour. The pitching upstream of the vanes is smooth and the object of this is to

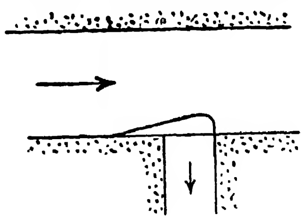


Fig. 19

keep the heavier grades of silt as low as possible. It seems somewhat doubtful whether this result is achieved. The eddies in a smooth channel are relatively weak and therefore a smaller proportion of sand is thrown up but the rise of .5 foot must cause eddies. If the pitching is necessary to prevent scour it should be level with the bed. "Corrective" vanes are added

downstream with the object of redistributing the heavy silt evenly and stopping the rotatory movement.

The following dimensions are recommended in the paper.

Width of head of distributary (Feet)	Dimensions shown in Fig. 18 (Feet)			Remarks
	hd	*ab*	*bc*	
2	4	2	4	The width *hd* must be considerable so that sufficient clear water may flow over the vanes to fill the distributary, with plenty to spare
4	5	4	4	
6	7	5	5	
8	8	6	6	
12	12	9	9	
20	18	13	12	
30	23	20	17	
40	28	25	22	

An alternative arrangement is to substitute for the vanes a low masonry wedge-shaped sill (Fig. 19) tapering at 1 in 4.

The Gagera branch, AF (Fig. 20) of the Chenab Canal, discharges some 5,000 c.ft. per second. Some years ago it was found to be silting. Its bed is level with that of the main canal. The bed of the right-hand branch AE, is lower. In order to remedy the silting, it was proposed to make a divide wall AB, extending up to above full supply level. This would simply have shifted the bifurcation from A up to B. The wall was actually made as shewn by the line AC, and of a reduced height. The length of the wall was about 450 feet. The top at the lower end was 8 feet above the canal bed but was stepped down, in the first 30 feet, to 3.8 feet. It then continued at that height throughout. The wall is reported to have done no good. After a few years it was extended from C to D — its total length being then 608 feet — and its height reduced to 2.7 feet. In this form it proved succesful.

It may be surmised that the sill or gates, along AE — required for limiting the water drawn off — obstructed some rolling silt which then found its way into the Gagera branch. It is probable that, with a depth of water of 8 or 9 feet, the wall ACD caused less heading up in the main canal than if it had been built on the line AF as an ordinary sill. Any such heading up would necessitate increasing the obstruction along AE and this would tend to stop more rolling silt and to reduce the benefit to the Gagera branch. It might appear that, instead of building the wall, it would have sufficed to increase the obstruction at EG and reduce it at GA so that rolled material would be swept over at GA. This however would have caused heavy action

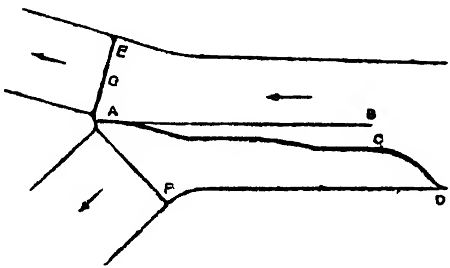


Fig. 20

on the floor downstream of GA. It may be doubted whether the wall AC actually did no good at all. It may have been merely insufficient. The conditions are not always exactly the same, either as to the silt brought down by the main canal or as to the silt-transporting powers of the branches. It has recently been found that silt deposit was occurring in the right-hand branch and the wall from C to D has been lowered one foot.

At the time the wall was built, the main canal, for a length of 3 miles upstream of the site, was scouring in the right-hand half of its channel and silting in the left-hand half. The cause of this is unknown. The channel is straight.

The measures described above for preventing or reducing silt deposit in branch channels, depend on the exclusion

from the branch of the heavier silt. Any such exclusion increases the silt in the main channel below the off-take of the branch. This may or may not be of consequence. The partial exclusion of heavy silt from irrigation distributaries by avoiding the drawing off of the lower water, has long been practiced. If, by the adoption of King's or other devices, an increased quanity of heavy silt is sent down the canal the effect of this will need consideration. It has needed it in the case of the Gagera branch.

A Canal with Headworks in a River. — In the case of a canal taking off from a river and provided with complete headworks — that is, a regulator across the canal head, and a weir, with under-sluices, across the river — it is possible to do a great deal more than merely cause the water to flow over sills and gates. The case of the Sirhind Canal, already referred to (Art. 1. and Chap. III., Art. 1), is a notable example. The canal (Fig. 21) is more than 200 feet wide, the full depth of water 10 feet, and the full discharge about 7,000 cubic feet per second. In 1893 when the irrigation had developed, and it became necessary to run high supplies in the summer — July, August, and part of September — the increase in the silt deposit threatened to stop the working of the canal.

In the autumn and winter, say from 25th September to 15th March, the river is low and the water clear and much of the deposit was picked up by it, but not all. In the five years 1893 to 1897 inclusive, the following remedial measures were adopted. Increased use was made of the escape at the twelfth mile [1]. This did some good, but there was seldom water to spare. In 1893 to 1894 the sill of the regulator was raised to 7 feet above the canal bed, and it was possible to raise it 3 feet more by means of shutters. This had little effect. The coarsest class of sand was $\overline{.4}$, and the velocity of the water, even of that part of it which came up from the river bed and passed over the sill, was over 2 feet per second, so that all sand was carried over. In 1894 to 1895 the divide wall, which had been only 59 feet long, was lengthened

[1] Some boats with scrapers (Art 1) were used in November 1893 when the escape was open.

to 710 feet, so as to make a pond between the divide wall

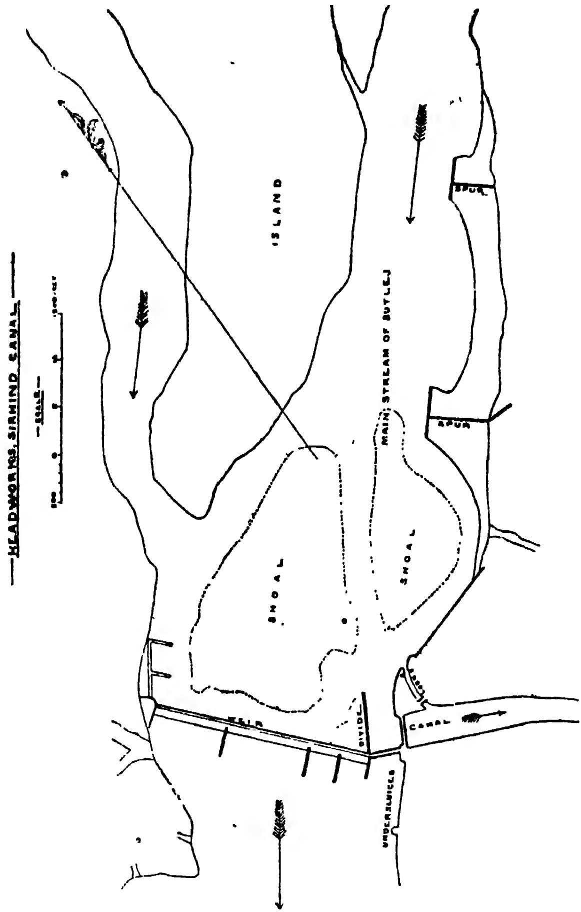


Fig. 21.

and the regulator but probably the leakage through the under-sluices — these are a continuation of the weir, in this

case not quite in the same line, between the divide wall and the regulator — was often as much as the canal supply, and the water in the pond was thus kept in rapid movement and full of silt. The canal was closed in heavy floods. This did some good, but probably the canal was often closed needlessly when the water looked muddy but contained no excessive quantity of sand. The above comments on the measures taken were made by Mr. Kennedy when chief engineer, The above measures did not reduce the silt deposits, but the scour in the clear water season improved, probably because higher supplies were run owing to increased irrigation. The deposit in the upper reaches of the canal, when at its maximum about the end of August of each year, was generally more than twenty million cubic feet. From the year 1900 a better system of regulation was enforced, the under-sluices being kept closed as much as possible, so that there was much less movement in the pond and much less silt in its water. By 1904 the deposit in the canal had been reduced to three million cubic feet, and no further trouble occurred except the necessity for occasionally closing the canal.

During the period from 20th September 1908 to 10th October 1908, the quantity of silt in the canal above Chamkour (twelfth mile) decreased from 19,325,800 cubic feet to 12,477,600 cubic feet. The quantity scoured away was 6,848,200 cubic feet. During this period no silt entered the canal. The quantity which passed out of the reach in question in suspension was 4,183,660 cubic feet, so that 2,664,540 cubic feet of material must have been rolled along the bed. The rolled material —probably mostly coarse sand — was 64 per cent. of the suspended material. During this period the escape, in the twelfth mile, was open, and the mean velocity in the canal just above the escape was about 4 feet per second, the depth of water being about 10 feet. The velocity near the escape was thus greater than the critical velocity for mixed silt,and even a long way up the canal it would be in excess of the critical velocity. The water seems to have carried about $\frac{1}{1800}$ of its volume of silt including rolled material.

As silt deposits in the pond, the velocity of the water

in it, along the course of the main current towards the canal, increases and eventually the water begins to carry coarse sand dangerous for the canal. In order to ascertain when this state of affairs has been reached, two methods of procedure are possible, One is to frequently test specimens of the water in the pond along the course of the main current and see when it contains more than $\frac{1}{16,000}$ of its volume of coarse sand. This plan would be troublesome and liable to error, and is rejected by Kennedy, who suggests that the depth and velocity of the water in the pond be frequently observed along the course of the main current. As soon as the velocity exceeds the critical velocity for mixed silt, it is time to close the canal and open the under-sluices and scour out the deposit from the pond. The period in which most silt is believed to have been deposited in the canal is the spring and early summer, say from 15th March to 1st July. This is the time when the snows are melting and the river water is clear but its volume great. It can then carry more sand than in the rains — 1st July to 15th September — when its volume is even greater but it is muddy.

Kennedy also suggested that some under-sluices should be provided at the far side of the river, that is, at the right-hand side of the weir. It would then be possible, by opening them, to let floods pass without interfering with the pond. The depth of the silt deposited in a great part of the pond amounted at times to 8 or 10 feet.

Silt trouble also occurred in the Chenab Canal which is bigger than the Sirhind Canal and discharges some 12,000 c.ft. per second. A remedy has been effected by lengthening the canal regulator and raising the level of the sill. The sand at Khanki — the head of the canal — is somewhat coarser than that at Rupar.

In the cases of the large canals — some have been mentioned above — taking off from the Indian rivers, the effect of the anti-silting arrangements on the river does not appear to have been considered. The river below the canal off-take has its discharge reduced by the canal draw-off. This alone may give it a tendency to silt. It also has to deal with the heavy silt which is excluded from the canal. The result,

in the case of the Punjab weirs and especially in that of the Sirhind Canal, is reported to have been a deposit of silt in the river downstream of the off-take. [1]

Art. 3. Bank Protection. The protection of a length of bank from scour may be effected by spurs, which are works projecting into the stream at intervals, or by a continuous lining of the bank. The latter may in some cases take the form of turfing or of planting or sowing rushes, osiers or the like. This particular protection, and others which will easily be recognised if they are not specifically mentioned, tends to promote the addition to the bank, by silting process, of material brought down by the stream. This is especially desirable when the protection is not undertaken until after some damage has been done. Spurs are also called "groynes" or "dykes". A spur forms an obstruction to the stream and when constructed, or even partly constructed, the scour near its end may be very severe, even though there may be little contraction of the stream as a whole. If the bed is soft a hole is scoured out. Into this hole the spur keeps subsiding, and its construction, or even its maintenance, may be a matter of the greatest difficulty. A high flood may destroy it. If it does not do so, it may be because the stream has, for some reason, ceased to attack the bank at that place. A continuous lining of the bank is not open to any such objection, and is generally the best method of protection. Spurs made of large numbers of rather small trees, weighted with nets filled with stones or sandbags, have been used on the great shifting rivers of the Punjab which swallowed up enormous quantities of materials The use of spurs on such rivers has now, in most cases, been given up.

If spurs are constructed over a long length of channel they are — at least until the spaces between them have become filled up, if that ever occurs — equivalent to a roughening of the channel and help to hold up the level of the water not only in low stages of the river but in floods. Spurs are best suited to a hard channel, that is, to gravel or shingle not to sand or earth.

[1] *Report on Sutlej Dam Project, 1919* Vol III (Nicholson).

If L is the length of a spur measured at right angles to the bank, the length of bank which it protects is perhaps 7 L—3 L upstream and 4 L downstream, — but the spur has to be strongly built, and its cost is, in many cases, not much less than the expense of protecting the whole bank with a continuous lining.

Whatever method is adopted, a plan, large enough to show all irregularities, should always be prepared, and the line to which it is intended that the bank shall be brought, marked on it.

Sometimes natural spurs exist as, for instance, where a tree projects into a stream or has fallen into it, and the holes between the spurs may be deep, so that a continuous protection would be expensive. Or there may be trees standing in such positions that, if felled, they will be in good places for spurs. In cases such as the above, spurs may be suitable even in a stream with a soft channel.

A fascine is a bundle — generally 4 to 6 inches in diameter — of long twigs, or even long coarse grass, bound together at intervals of about 2 feet with twine or wire. A mattress is a broad, flat structure generally composed of layers of fascines, those in one layer running longitudinally, in the next layer transversely. It is generally flexible. Mattresses are also made by weaving willow branches together. A stiff mattress can be made by adding poles or bamboos.

Spurs. A spur may be made of: —

(*a*) Loose stone.

(*b*) Mattresses weighted with gravel or stones.

(*c*) Earth or sand closely covered with fascines, or with stone — rubble or boulder — pitching.

(*d*) A double line of stakes with fascines or brushwood laid between them (Fig. 22).

(*e*) A single line of stakes with planking or basket work on its upstream side, or with twigs or wattle laid horizontally and passed in and out of the stakes, as in Fig. 35.

(*f*) A single tree with the thick end of the trunk on the bank and with stakes, if necessary, to prevent the current from moving it.

(*g*) A number of small trees heaped together and weighted with nets full of stones or sand-bags.

(*h*) A layer of poles and over them a layer of facines on which are built walls of fascine work so arranged as to form cells or hollow rectangular spaces which become filled with silt.

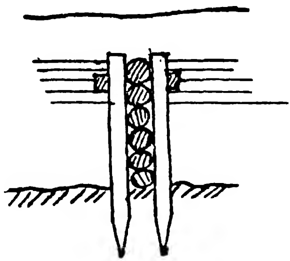


FIG. 22.

(*i*) Mattresses running out into the stream and having their inner ends staked to the bank while the outer ends float, other mattresses being added over them and projecting further into the stream, and the whole eventually sinking.

Of the above methods of construction, (*d*) and (*e*) are suited to somewhat small streams. Some other methods will be mentioned below.

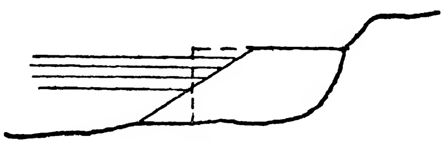
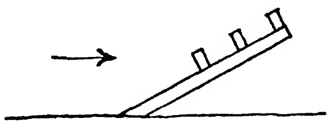


FIG. 23.

Spurs made of earth are generally carried up to above high flood level; otherwise they would probably be damaged or destroyed in floods. For other kinds of spurs the height varies according to circumstances and judgment. The end of a spur is generally sloping (Fig. 23). It then causes less disturbance than if built to the form shown by the dotted line.

Instead of running out at right angles to the bank a spur may be inclined downstream. This reduces the eddying and scour round the end. The ends of a system of spurs should be in the line which it is intended that the edge of the stream shall have (Fig. 24). When a spur is long it may have small

subsidiary spurs (Fig. 25) to reduce the rush of water along it; or its end may have to be protected in the same manner as the advancing end of a closure dam (Chap. V., Art. 2).



Or it may in a large river be T-headed (Fig. 53, Chap. V., Art. 3). The particular spur there shewn is of earth pitched with stone. The head may even be entirely of stone.

In large rivers, particularly during floods, much trouble is apt to be caused by the strong swirls (Fig. 26) which are set up downstream of the spurs. The current near the bank is reversed and severe erosion of the bank may occur. The occurences of severe swirls may indicate that the length of the spurs should have been reduced and their number increased. In 1909 the Indus was eroding its right bank and threatening to destroy the town of Dera Ghazi Khan. A clump

of date palms formed a promontory and resisted erosion to some extent. A suggestion was made — by an engineer of eminence who had formerly been consulted in the case — to the effect that the date palms be removed, the reason given being that they caused disturbance and scour. On this principle it seemed that spurs would have to be made, not to

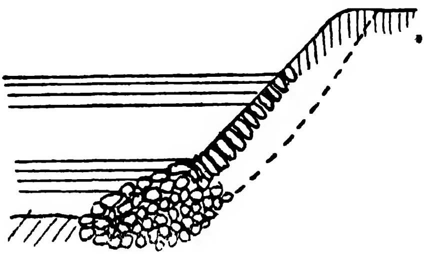


Fig 27

protect a bank but to cause it to be eroded. The local engineers did not adopt the suggestion. In course of time the trees fell into the river and disappeared. The remedy for swirling and disturbance is, not to remove the spur but to protect

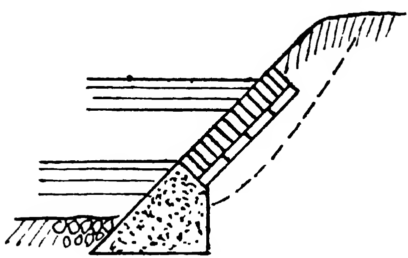


Fig. 28.

the portion of bank which is attacked, by lining it or by small additional spurs or by "Brownlow's weeds" mentioned below.

Continuous Lining of the Bank. — When, instead of spurs, a continuous lining of the bank is adopted, the work generally extends up to about high water-level or even above it, so as to be a protection against waves. The lining of the bank may be stone or brick pitching (Figs. 27 and 28), loose

stone (Fig. 29), fascines (Fig. 30), turfing, plantations, brushwood, or other materials laid on the slopes. Before protecting a bank it is best to remove irregularities and bring it to a regular line. This can be done by filling in hollows or by cutting off projections. The latter method gives the best

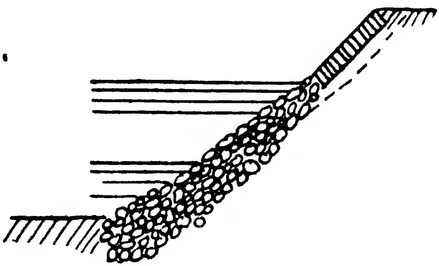


Fig. 29

foundation for heavy work. It is also desirable to make the side slope uniform. Where the slope is as shown by the dotted lines in Figs. 27 to 29, filling in can be effected, but cutting away the upper part of the slope is also feasible. Such

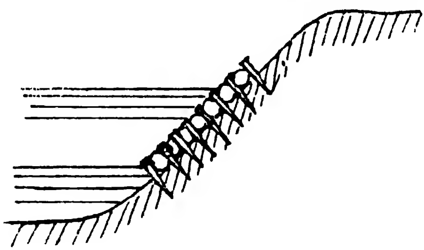


Fig. 30

cutting away has been proposed as a remedy in itself in cases where the steep upper part of the slope was falling in, but it is likely to be only a temporary remedy. Any earth filling must be well rammed in layers.

Pitching may rest, if boats are required to come close to the bank, on a toe wall of concrete, as in Fig. 28, or otherwise

on a foundation of loose stone, as in Fig. 27. When concrete is used the bed is dredged to such a depth as will provide against undermining by scour. Sloping boards attached to piles are placed along the front face and the concrete is dropped in under water.

When loose stone is used, dredging is not necessary, but the stone is allowed to gradually sink down and more is added at the stop. A certain proportion of the stones should be of large size.

The slope of stone or brick pitching is usually from 2 to 1 to 1 to 1, but it may be as steep as ½ to 1. The earth behind the pitching must be well rammed in layers. In order to prevent the earth from being eaten away by the water which penetrates through the interstices of the stone or brick, a layer, 3 to 6 inches thick, of gravel or ballast in placed over the earth and rammed.

Sometimes pitching consisting of a solid concrete slab is used on rivers at bends. It goes down to low water level and rests on thick wooden piles.

When fascining is used, the fascines can be laid on the slopes (Fig. 30) and secured by pegs driven in at short intervals, between the fascines.

Sometimes the pitching or loose stone is not carried up to the top of the bank, or even up to high flood-level, and the bank above the pitching is protected by turfing — the pieces of turf being placed on edge normally to the slope if very steep (Fig. 29) or laid parallel to the slope if it is not very steep—or, above ordinary water-level, by plantations of osiers or willows which obstruct the water and tend to cause silting, and whose roots bind the banks together.

Another method of using fascines is to lay them on the slopes with their lengths normal to the direction of the stream. The upper end of a fascine is above low water, and the lower end extends down to the bed of the stream. Sometimes large ropes made of straw, or rough mats made of grass, are laid on the slopes and pegged down.

A deep recess in the bank (Fig. 31) can be filled in, before the protection is added, with earth well rammed. On the

Adige the filling material consisted (Fig. 32) of baskets filled with stones, small spurs being made at intervals, as shown by the dotted lines, to arrest flood water and cause it to deposit silt, At the back of the berm, poplar or willow slips were planted, and these grew up and their roots held the bank together. This system succeeded well.

Protection of the banks of the Mississipi against erosion or "caving" has, for long, been systematically carried on. The

FIG. 31

usual protection is a mattress. At first the mattresses were thin and made of willows woven as in basket work — a common width was 250 to 300 feet and the length might be 800 feet — but they were afterwards thicker and made of fascines bound with wires, rubble stone being laid on then as in the case of the Missouri described below.

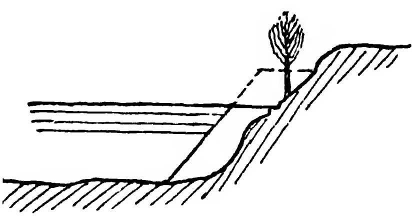


FIG 32

On the Mississippi below Natchez a very high bank, subject to erosion, was protected by a series of cells somewhat similar to those described above as in use for spurs. Each cell was of poles and brushwood and was roofed over. Just above low water level there was a layer of hard conglomerate. On this was laid a 12-inch mattress, the bundles of brushwood being bound at every 4 feet by galvanized wire. The mattress was anchored at every 50 feet by ½-inch galvanized wire rope. On the mattress a series consisting of

5 rows — parallel to the stream — of cells was built. The total width was 28 feet. A second tier of 4 rows was built over all except the front row, and so on, so that there was a series of 5 steps facing the stream Three years after the completion of the work the cells had partly silted up and the work was covered by a series of reinforced concrete slabs and was protected by a toe wall [1].

In the Mississippi delta, owing to the increasing amount of bank protection being undertaken and to the large inroads on the supply of willow for mattresses, articulated mats are being made of reinforced concrete, 3 inches tick. The reinforcement is electrically-welded wire. Each slab is about 4 feet square, and 25 slabs form a mat 100 ft. × 4 ft. The mats

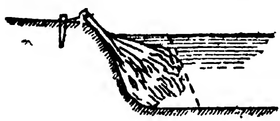


FIG 33

are constructed on a line of moored barges and are then taken on board a mat-sinking barge which carries launching ways and a locomotive crane [2].

A method of protection which is excellent when the water contains heavy silt and is not too deep, is what is known on Indian canals, as "bushing". Large leafy branches of trees are cut and hung, as shown in Fig. 33, by ropes to pegs. They must be closely packed so as not to shake. At first they require looking after, but silt rapidly deposits and the branches become fixed and no longer dependent on the ropes. If the work is carefully done, the result is a smooth, regular and tenacious berm, as shown by the dotted line in the figure, Bushing is used chiefly in places where the bank has already been partly eaten away. On a large channel whole trees, of moderate size, can be used, branches being added if necessary. Bushing is not suitable if there is a weir

[1] *Trans Am Soc. C E* Vol 84, p 303.
[2] *Soc Am Military Engineers*, Vol. 15, p. 215.

or regulator a short distance upstream because the agitation of the water prevents silt from being deposited.

Another method, also used on canals, is to make up the bank with earth and to revet it with twigs or reeds, as shown in Fig. 34. The foundation must be taken down below bed level, otherwise the work may slip. This kind of work is done when the canal is dry. It is suitable even immediately

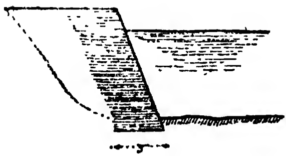


FIG. 34

below a weir or regulator. It is more expensive than bushing.

If the bank consists of sand or of very sandy soil, it must in any case have a flat side slope such as 3 to 1. If the sand is in layers alternating with firm soil, and the slope say 1 to 1, it is a good plan to dig out some of the sand and to replace it by jamming in clods of hard earth.

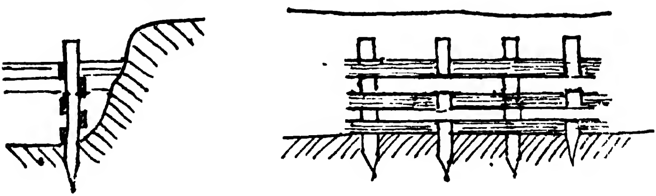


FIG. 35

Staking (Fig. 35) may be used, the stakes being one or two feet apart from centre to centre, and long twigs laid horizontally being passed in and out of the stakes, or small brushwood filled in behind the stakes. The stakes can be driven parallel to the slope of the bank instead of being vertical. They can also be driven very close to each other. Planks on edge can be used, being nailed to the stakes — which in this case are sometimes squared — so as to form a continuous lining.

Staking is best suited to small streams. It is liable to be forced out of line by earth from the bank pressing against it. On the large silt-bearing Indian canals bushing is far better and is less expensive.

The Villa system of bed protection (Art. 1) has also been successfully used for bank protection on the Scheldt, and on the Brussels-Ghent Canal, the prisms being about 19 × 10 × 4 inches, and having overlapping joints. The bands of prisms are placed in position by a boat or pontoon, the bands unrolling over a drum, The boat is provided with an oscillating platform carrying rollers at its end. A thin layer of gravel is laid over the bank and is pressed down by the rollers before the prisms are laid on it [1]. In Japan, and on the river Miami in America, concrete blocks, about 2 × .5 × .5 feet (Fig. 36), have been similarly used for soft banks. The wires

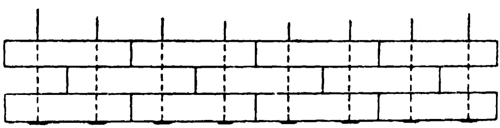


FIG 36

or wire ropes, shown by the dotted lines, are placed in position and the lowest layer of blocks threaded on, the others following.

In the case of the river mentioned in Chap. VIII., Art. 6, where extremely high velocities were met with, cylindrical rolls of wire-netting were made, each 50 feet long and 5 feet in diameter, and filled with boulders. These rolls can be used for bank protection. The netting was made by wires 6 inches apart, crossing each other at right angles and tied together at the crossings by short pieces of wire.

On ship canals a berm (Fig. 37) is frequently made a few feet below the water-level. It serves as a foundation for the pitching, which need not usually extend down to more than 5 feet below the water-level. Below that the wash has little or no effect on the banks. On ordinary navigation canals

[1] *Proc Inst C. E.*, Vol CXXXIV, and Vol. CLXXV

a similar berm is sometimes made — one or two feet in width and a foot or less below the water-level — and rushes are planted on it.

Sometimes a bank has been protected by what were originally known as "Brownlow's weeds", consisting of bushes or branches of trees attached to ropes. The end of the rope is fastened to the bank and the weeds float in the stream alongside the bank, casks being added, when needed, to support them.

To protect a bank from ice, which exercises an uplifting force on pitching, use has been made of a covering of a kind of reinforced concrete consisting of slabs of concrete with wires embedded in it, and fastened down by wires, 20

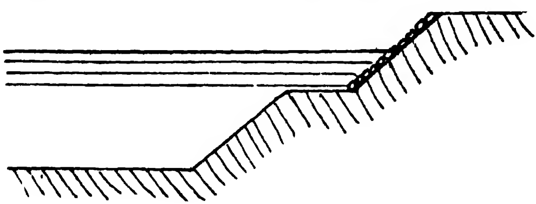


Fig. 37.

inches long, running into the bank, these wires being embedded in mortar so as to act like stakes.

Retaining walls, whether vertical or having a batter, may be necessary in special circumstances, as where space is restricted owing to buildings etc. The principles for constructing them are the same as for other retaining walls.

Works on Some Large Rivers. — Protection work, extending over miles of bank, has of late years been carried out on the river Missouri where the stream was eroding its bank, and threatening a railway bridge and also the line of railway which skirts the river. Short spurs — also called "dykes" — were constructed of stone ("rip-rap"). These projected a moderate distance into the stream. Longer spurs called "retards" were formed at a somewhat acute angle with the downstream bank. They were rafts formed of logs, old telegraph poles and tree tops, looped about with wire rope.

The lengths ranged from 50 to 300 feet — in one case 450 feet. When short they were anchored to the bank, when long to concrete piles sunk by means of a water jet till the top of the pile — to which a cable was secured — was 10 feet below the river bed. The raft was finally tilted into an inclined position, the upstream edge being sunk to the bottom, the downstream edge standing up above water.

With both kinds of spurs the swirling of the water on the downstream — and sometimes on the upstream — side, especially during floods, caused erosion of the bank which was stopped by lining the bank with mattresses. The mattresses consisted of fascines made of willow and fastened together by ⅜-inch galvanised steel cablees. The mattress commenced from a point on the bank some 10 feet above ordinary water-level and extended well out into the stream. It was then sunk by stones and completely covered by a revetment of the same. When erosion was severe it was necessary to stop it — pending the laying of the mattress — by stretching a cable across the embayment and attaching tree trunks and tops to it [1].

The accompanying illustrations are views taken respectively from the upstream and downstream sides of the same retard.

On the same river and to protect the same railway — as well as other sites controlled by municipalities and private owners — spurs of considerable length and sloping downstream as before, have been made by stretching a wire cable from the shore to a concrete pile sunk in the river as before. To the cable are attached trees and branches which hang in the stream, or "A-frames" which support the cable and are covered with netting on their upstream sides. A spur of this kind of course allows water to pass through it and silt deposits on its downstream side.

In other spurs the cables have been attached to spider frames — that is, three long bars of angle iron crossing each other centrally and at right angles — placed in the stream

In sinking the concrete piles a water-jet pile is employed.

[1] *Railway Review*, Chicago. 13th Sept. 1919.

The latter is from 20ft. to 40ft. long and 16in. or 14in. square. A 4-in. pipe extends through the centre nearly to the point, and has ½-in. lateral branches, spaced 30in. on centres, which end in elbows turning upward along the sides of the pipe. A 2-in. pipe having a separate hose connexion extends through

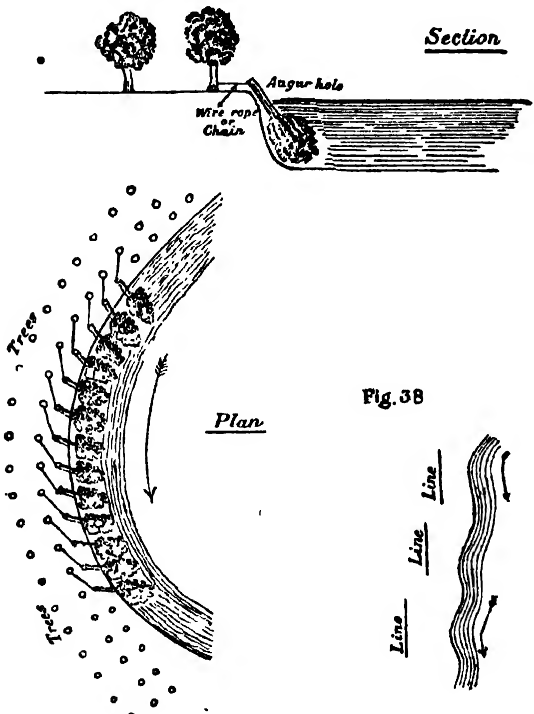


Fig. 38

to the point. In some cases this pipe is omitted and the 4-in. pipe extends to the point where it terminates in a 2-in. nozzle. Thus, in addition to the boring jet, there are streams of water flowing upward along the sides of the pile.

The protection of banks by bushing and small trees has been mentioned above. A precisely similar arrangement could

be made on a large and deep river if larger trees, with their roots upwards, were to be used. A little work of this kind was done on the Indus in 1892 and also on the river Markanda in the Punjab. The works failed because of the insufficient size of the trees used. Large trees could not be hauled several miles nor be obtained in anything like sufficient numbers. Planting three or four rows of trees, roughly parallel to the bank of a river and at such a distance from it that they are likely to attain a large size before the river attacks them, is a plan which may be worth trying, especially in places where trees grow quickly and where their presence is convenient. The rows can be about twenty feet apart. When about to be attacked the first row would be loosely chained to the second (Fig. 38). The first row, where attacked, would fall in but the trees could not be carried away. If the second row showed signs of falling — they would not all fall — they could be chained to the third row. A possible arrangement over a long reach is shown in the lower figure on the right.

For the special protection to banks required near weirs, bridges and other works see Chap. VIII, Art. 6.

CHAPTER V

WORKS FOR THE CONTROL OF STREAMS

Art. 1. Diversions or Cut-offs. In America the term "diversion" is used for an artificial off-take from a river. In England it is commonly used for the case where water is taken off from a channel and turned into it again. The new course is generally shorter than the old one, and the diversion is then often called a "cut-off." Suppose a cut-off to be made and the old channel to be closed. The first result

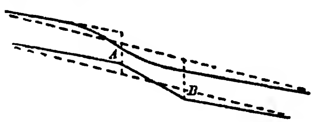


Fig. 39

is a lowering of the water-level upstream and a tendency to scour there, and to silt downstream of the cut-off. Fig. 39 shows the longitudinal section of a stream after a cut-off A B has been made. The bed tends to assume the position shown by the dotted line. This line — if the channel was originally only just stable — meets the line of the old bed at points so far upstream and downstream that the new slope may differ inappreciably from the old. In other cases some idea of the position of the new line can be formed by considering the velocity which the soil will stand with reference to the depth of water (Chap. VI Art. 6). Unless the stream in the upstream reach meets with hard material it is likely to scour for a long distance. Silting in the downstream reach generally occurs. The abolition of the three bends (Fig. 40) will be equivalent to some increase of smoothness in the channel.

If both the diversion and the old channel remain open, the water-level at the bifurcation A, will be lowered still more, and the tendency to scour above A increased and below A reduced. The water-level at B is the same as before except for any loss of water which may occur in the old channel.

If the material is soft enough to be scoured by the stream, it is often practicable to excavate merely a "leading cut", that is to excavate the diversion to a small section and to let it enlarge itself by scour. This operation is immensely

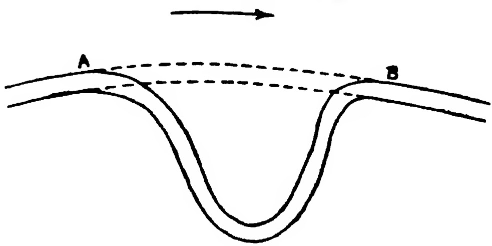


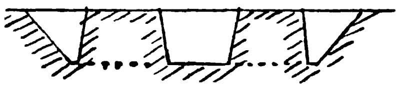
Fig. 40.

facilitated if the old channel can be closed at the bifurcation. The question whether the scoured material will deposit in the channel downstream of the diversion must be taken into consideration; also the question whether the diversion will continue to enlarge itself more than is desirable. The velocity in the leading cut is a maximum if its section is of the "best form", that is if its bed and sides are tangents to a semicircle whose diameter coincides with the water surface. This may not be the section which will give most silt-supporting power, though it probably gives a maximum of rolling power.

In order to prevent the enlargement of the cut taking place very irregularly, the excavation can be made as shown in Fig. 41, water being admitted only to the central gullet. If, in any reach of the cut, the scour takes place chiefly to one side of the central gullet, say the right, the stream may establish communication with only the right-hand gullet

and then tend to scour to the right of that again. To enable such action to be controlled, short portions of the side gullets can be left, at short intervals all along them, unexcavated. Any such portion can, when scour of the central gullet is in progress, be left to act as a spur or can be removed. Other means for temporarily protecting the slopes of the side gullets can be adopted if necessary.

If a diversion is made, not with the object of lowering the water-level, but merely in order to shorten the channel, the increased velocity caused by the steepened slope may



be inconvenient. In this case a weir or weirs can be added (Art. 3, p. 125).

If the water contains sufficient silt to enable the abandoned loop to be silted up within a reasonable time, it may be desirable to do this. The silting up may, for instance, increase the value of the land. The loop should be closed at its upper end. Water entering the lower end will cause a deposit there. When the lower end is well obstructed by silt, the upper end should be opened.

In river diversion works, spurs — described in Art. 3 — are sometimes used to "drive the river" down a branch channel. A spur may make the current set against the branch head, but unless the spur is so long as to greatly contract the water-way, the rise of water-level will not be great, except in cases of high velocity, and the river will continue to distribute itself according to the discharging capacities of the two branches. With ordinary velocities it is only by closing or thoroughly obstructing one branch or enlarging the other over a great length, that the stream can be forced to alter its distribution of discharge.

Two of the branches or "passes" into which the Mississippi divides before it enters the sea, were partially closed in order to increase the discharge of the remaining branch. Mattresses

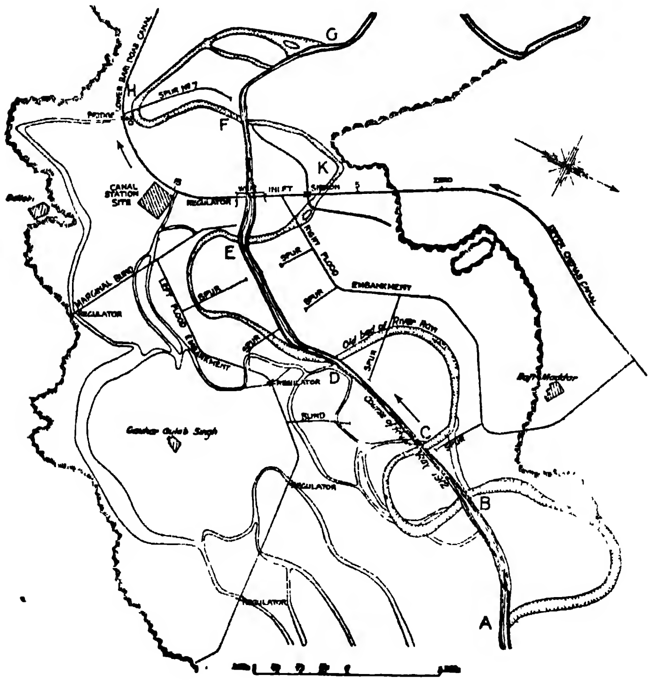
weighted with stones were used. It would appear that they must have been of very considerable thickness, in the aggregate, before they effected much reduction in the discharges.

It is sometimes said that it is easier to lead a river than to drive it. This remark is probably based on the fact that spurs, such as those considered above, may produce little effect, whereas the excavation of a diversion or the deepening of a branch by dredging it, is more likely to produce some result. There is, however, no certainty about this. Sometimes too much is expected of such channels. Calculations are not always made as to the scouring power of the stream, nor is account always taken of the fact that as the cut scours its gradient flattens.

The Indus had been cutting into its right bank and had formed a bend. During the low water season when it was about 20 feet deep, an attempt was made to divert it by a straight cut, about a mile long, across the sand-bank in the bend. Owing to the high level of the sub-soil water, the cut could only be dug down to about 2 feet below the water-level of the river. The slope of the cut was about one-and-a-half times that of the river, but owing to the small depth of water the velocity was low, and the cut, or at least its upper part, rapidly silted up. A reason given for its failure was that there was no set of the stream against its head. Any such set might have given an inch or two more water, and the cut might have taken a few days longer to silt up (p. 135).

A very remarkable diversion of the river Ravi was effected, under the direction of Sir John Benton, below the level crossing where the Upper Chenab Canal (Fig. 42) enters the Ravi and the Lower Bari Doab Canal leaves it. The diversions AB, BC, CD, DE, were effected by means of leading cuts, the flood waters being also directed by the spurs. Along the diversion EF three parallel 150-foot leading cuts were made. In the first flood they widened within a week, to an aggregate width of 1,682 feet although the old loop E K F was open. The loop largely silted up. The diversion F G formed itself, as had been intended, on the

first arrival of the flood above mentioned, without any leading cut and although the spur No. 7 had not been constructed and the loop F H G was open. The water-level in the Ravi at F was high enough to flood the land along F G, and the soil was sandy. High velocity of approach had



everything to do with the success of the operation. The flood water, held up owing to the narrowness of the leading cuts, and coming down the long slope of the weir at a high velocity, scoured a channel along F G with great rapidity. The fall in the river bed from F to G was about 10 feet [1].

[1] *Proc. Inst. C E, Vol* CCI p. 32

The embankments marked "spur" come into use only when there are floods. They confine the course of the floods to the river channel and prevent the opening out, during floods, of new channels. Such embankments are made of earth, only the heads, which are close to the channel, being protected by stone pitching.

On the Mississippi, cut-offs in the upper part of the river were found to be harmful. If they were not also made lower down, they might cause silting in the lower reaches and raising of the flood level there. On the old main line of the Western Jumna Canal — a tortuous channel — some cut-offs caused raising of the bed for many miles lower down, so that bridge arches which were formerly wholly above water became submerged all but the crowns.

It has been seen (Chap. III., Art. 4) that if a tortuous channel has attained average stability, it may shorten itself by short-cuts, and that it can then more readily get rid of floods but loses its stability. The probable effect of a series of diversions in a long length of such a river is considered — and diversions further dealt with — in Chap. VII. Art. 4.

As regards banks, side-slopes and other matters the information given in Chapter VI applies of course to diversions.

Art. 2. Closures of Flowing Streams. The closure of a flowing stream by means of a dam is usually attended with some difficulty and sometimes with enormous difficulty. There may be little trouble in running out dams from both banks for a certain distance, but as soon as the gap between the dams has become very much less than the original width of the stream, the water on the upstream side is headed up and there is a rush of water through the gap, which tends to deeply scour the bed and to undermine the dams. The smaller the gap becomes the greater is the rush and scour.

The closure is most easily effected at or near to the place where the stream bifurcates from another. Then, as the gap decreases in width, some of the water is driven down the other stream and it does not rise so much. Eventually all the water goes down the other stream, and the total rise is only so much as will enable this other stream to carry the

increased discharge. If the closure is not effected near a bifurcation, the rise of the water will go on even after the closure is completed, and it will not cease, unless the water escapes or breaks out somewhere, until it has risen to the same level as that to which it would have risen if the closure had been at the bifurcation, or perhaps not quite to the same level, since there may still be a slight slope in the water surface and a small discharge which percolates through the dam or into the soil. Sometimes in such a case it is possible to arrange for temporary escapes or bifurcations — at the dam — which will be shallow and therefore easily closed, after the main closure has been completed.

A closure is, of course, far more easily effected where the bed is hard than where it is soft. Very often it is best to close temporarily at such a place or near a bifurcation, even if the permanent dam has to be elsewhere, and then to construct the permanent dam in the dry channel, or in the still water, and remove the temporary one or cut a gap in it.

Generally the best method to adopt in a closure is to cover the bed of the channel beforehand — unless it is already hard enough — with a mattress or floor, such that it cannot be scoured as the gap closes. A floor may consist of a number of stones or sandbags, dropped in from boats or by any suitable means, and placed with care so that there shall not be gaps or mounds. Sandbags should be carefully sewn up. A mattress may be made of fascines laid side by side and tied together, floated into position, weighted and sunk. Even a carpet made of matting or cloth and suitably weighted has sufficed in some cases. If the scour is likely to be such that stones or sandbags will be swept away, the stones may be placed in nets, baskets, or crates. Sandbags may also be placed in nets. The long rolls of wire-netting filled with stones, described in Chap. IV., Art. 3, or those described below, can be used where velocities are very high. The nets, baskets etc. can be slung from a derrick on a barge. The floor or mattress need not usually extend right across the stream. It must cover a width much greater than — perhaps twice as great as — the width of the gap is likely to be when scour

begins. Its length, measured parallel to the direction of the stream, must be such that severe eddies in the contracted stream will have ceased before the stream reaches its downstream edge. It need not extend to any considerable distance upstream of the line of the dam.

The dams when started from the banks can generally be of simple earth or gravel, or loose stone, but before they have advanced far they will probably require protection at the ends by sandbags or stones, or by staking and brushwood, or by fascines. As soon as the dams have advanced well on to the mattress and their ends have been well protected, it is best to cease contracting the stream from the sides and to contract it from the bottom by laying a number of sandbags across the gap so as to form a submerged weir. In this way the rush of water is spread over a considerable width of the stream. The weir is then raised until it comes up above water. Leakage can be stopped by throwing in earth, or gravel, or bundles of grass on the upstream side. Sometimes it is best to construct the mattress over the whole width of the stream, and to effect the closure entirely by a weir, carrying each layer right across before adding another. The banks of the stream, if not hard, can be protected by sandbags, stones, staking and brushwood or fascining.

The chief cause of failures of attempts to close flowing streams is neglect to provide a proper floor or mattress. The stones or other materials may be of insufficient weight or not closely laid, or the extent of the floor may be insufficient. In a soft channel and deep water loose stones in almost any quantity may fail unless a mattress of fascines is laid under them. Another cause of failure is running short of materials, such as sandbags. Allowance should be made for every contingency, including making good any failure of parts of the work. Enormous sums of money have been wasted, and vast inconvenience, loss and trouble incurred, in futile attempts to close breaches in banks, or gaps in dams.

Sometimes the gap is closed by sinking a barge loaded with stones, or by sinking a "cradle" or large mattress made

of fascines, taken out to the site by four boats, one supporting each corner, and then loaded with stones and sunk. Another method is to run out a floating mattress of fascines from one side of the gap to the middle and sink it, then to proceed similarly on the other side, and so on.

In India closures of streams having depths of 6 or 8 feet are effected by means of rough trestles made from trunks of small trees and placed at intervals in the stream like bridge piers, one leg of the trestle inclined up stream and one downstream. Each pair of adjacent trestles is connected by a number of rough, horizontal poles. Against these are placed bundles of brushwood. Earth is at the same time collected and is rapidly added at the last. The chief danger is the undermining of the bed by scour. This is prevented by driving in stakes and placing brushwood against them. Closures of small channels or of breaches in the banks of canals are effected by means of staking and brushwood. Where dangerous breaches are liable to occur, it is a good plan to have a barge, fitted up with a small pile-driver and carrying a supply of sheet piles, ready at a convenient spot.

In any case in which the provision of a proper mattress has been omitted, or when the mattress has been destroyed, or when a breach has occurred in an embankment, whenever, in short, it is evident that the gap cannot be closed until some other escape for the water is provided, it may be possible to provide such an escape by cutting partly through the dam or embankment on the downstream side at another place, thoroughly protecting that place and extending the protection downstream and away from the dam or embankment. The water can then be let through, and the closure of the old gap attempted. If a closure is effected, the protected gap can then be closed. Sometimes it may be desirable to make such a protected gap beforehand and with deliberation.

Dams for closing streams which are dry can be made similarly to flood embankments (Chap. VII., Art. 4). Sand does very well, provided it is protected by a covering of clay or by fascining.

Instances of Closures of Streams. — In 1904 the Colorado River broke into the Salton Sink—a valley covering 4,000 square miles. Unsuccessful attempts were made to close the stream by two rows of piles with willows and sandbags between them, by a gate 200 feet long, supported on 500 piles, and by twelve gates each 12 feet wide. A "rock-fill" dam was then constructed on a mattress 100 feet wide and 1.5 feet thick. More than 1000 men worked on it. The river, which was 600 feet wide, broke through, but was stopped

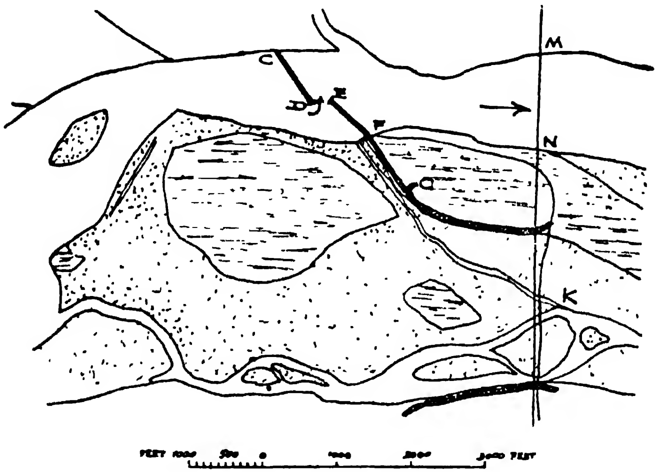


FIG 43

by the construction of three paralell rock-fill dams in the gap [1].

At the site of the railway bridge over the river Tista in Bengal, it was necessary to close the main stream (Fig. 43), which flowed at the left side of the channel, while the bridge had been built at the right. The bed was of sand, width 500 feet, depth 6 feet, and discharge 3700 cubic feet per second. The first attempt to close the stream was made at M N, a floor of stone 200 feet long, 20 feet wide, and 2 feet thick, being laid in the middle of the stream, and dams of earth,

[1] *Proc. Inst. C. E.*, Vol. CLXXI, p. 360.

sandbags, and stones being run out from each bank. As the gap decreased in width the bed was torn up and the work failed. The heading up was 3 feet 9 inches. It was recognised later, that the site should have been at the bifurcation higher up, and that the stone floor should have been laid on a mattress.

In the next working season the dams C D and E F G were made. The dam C D was of earth. Two walls, each consisting of a double line of bamboos with the spaces between the lines filled with bundles of grass weighted with earth, were run out 50 feet in advance of the earthwork near the lines of the toes of the slopes. Along the line of the upper wall a mattress of broken brick 10 feet in width, and 1 foot thick, was laid, and was kept 50 feet in advance of the wall. A total length of 1000 feet of embankment was made in five months and pitched on its upstream side. The end was strongly protected by a mass of stone. The embankment F G was of earth. The dam E F consisted of three lines of piles driven 10 feet into the bed. A mattress weighted with stones extended for 20 feet upstream of the dam and 40 feet downstream. A gap of 150 feet was left at D E, and was not protected by a floor of any kind. A channel, parallel to F G and extending to K, had been dug to a width of 200 feet. During the floods the heading up at D E was about 2.5 feet, and the water was 30 feet deep. The line E F was greatly damaged and was repaired.The cut F G K gradually enlarged, and by the end of the floods more water was going down it than down the main stream. The gap D E was finally closed by means of a line of bamboos and grass, the bed being protected by a carpet, 100 × 50 feet, made of common cloth weighted with sandbags. The success of the operations turned on the scouring out of the cut F G K . It is remarkable that the gap D E did not become wholly unmanageable in the floods [1].

A method adopted by Beresford for closing side channels of the Nile, where the depth of water was 13 feet and the current velocity 6 or 7 feet per second, was to lay down carpets

[1] *Proc Inst. C E*, Vol. CL

of sail-cloth 400 to 500 feet long and 70 feet wide, from a bridge of boats anchored upstream. The carpet protected the bed while the channel was closed with rubble [1].

In order to construct a long spur across a side-channel on the left side of the Sutlej at Rupar, it was decided to temporarily close the channel. Its width was 160 feet and its discharge some 4,000 c.ft. per second. The channel was mostly of sand. Its right bank was an island. A mattress of wire netting, 4-inch mesh — 14 B.W.G. — covered with brushwood and 50 feet long, parallel to the stream, was laid on the bed. A bridge was then constructed, the piers and abutments being of concrete blocks. The piers were 30 feet apart, centre to centre, and the superstructure was of steel girders. Boulders were laid on the mattress and when the stream began to sweep them away, rolls of the netting — — 8ft. × 4ft. and 4ft. × 4ft. — were placed on the bridge, or on platforms projecting from barges, filled with boulders and rolled off. These rolls were not shifted by the stream. Sleepers were stood on end — like the needles in a dam — resting against the rolls and against the bridge girders, and sacking spread on their upstream faces. Earth was brought on to the bridge in trucks to complete the dam [2].

The plan of throwing a temporary bridge across a stream, as a preliminary to closing it, has been adopted in more than one instance of late years — besides the case mentioned above and for reasons other than those given — and may be considered to be generally the best method. In the absence of a bridge it may be impossible to collect the materials quickly enough at the final closure. In the case of the closure of an arm of the river Dharlla in Bengal, in connection with a railway bridge [3], the temporary bridge was of piles and planks, each pier consisting of two piles (Fig. 44). The width of the stream had been 400 feet but was reduced, by means of banks of earth and sandbags, to 250 feet. The discharge was 3,500 c.ft. per second. In order to prevent the sand-bags

[1] *Proc Inst. C E*, Vol CCIII p 301.
[2] *Proc Punjab Engineering Congress* 1915
[3] *India Railway Board Technical Paper*. No 142.

— when used in the closure — being carried away, bamboos were driven into the bed of the stream at one-and-a-half-foot intervals over the whole area to be covered by the mattress. The bridge platform gave sufficient headway to admit of the construction of the mattress underneath it. Seven strong ropes, made on the spot, of raw jute fibre and supported by slings, were stretched across the stream under the bridge. Across them were laid bamboo battens 2 feet apart. On this framework was constructed the grass mattress, 250ft. × 30ft. × 2ft. The sand-bags were stacked on the bridge. Downstream of the bridge a carpet of bricks, 30 feet wide and 2 feet thick, had been laid in order to prevent the stream from deepening.

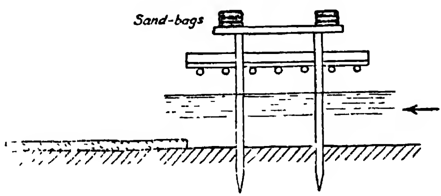


FIG. 44

The mattress was weighted and lowered and the sand-bags thrown in. This work was began at 7 a.m. By noon the closure had been effected. In one place a stream 30 feet wide, had formed under the mattress but it was closed by bamboos and sand-bags, the brick carpet being largely instrumental in saving the situation

In the case of the closure of an arm of the Sutlej near the Empress Bridge[1] the method adopted was similar to the above but the work was heavier The width of the stream had been 475 feet and was reduced to 195 feet, but its depth was 12 feet and its discharge 7,000 c.ft. per second. Each pier of the temporary bridge was of 3 piles at 10 feet centres. There were 13 spans, each of 15 feet. Bamboos were found to bend and give trouble Instead of them, 2-inch iron pipes were used. The downstream carpet was of stone and was

[1] *Proc Punjab Engineering Congress* 1916.

30 feet wide and 2 feet thick. During the progress of the work freshets came down and increased the discharge to 9,000 c.ft. per second. The sand-bags were not all ready and the bed was scoured somewhat. The stone carpet sank 6 feet. Eventually the bags were stacked on the bridge and the closure effected. Only a small proportion of the pipes could be recovered. It was concluded that with so heavy a discharge as 7,000, c.ft. per second, a tramway is necessary in order to carry the bags to the work with sufficient rapidity, and that in any case the mattress should be sunk evenly and covered all over with bags before the mass of the bags are added, and that the work should then be raised evenly from end to end. Also that a sand-bag should be as big as one man can carry and should not be filled so full as to be hard. The loose parts fill up spaces. Some bags should be half-size for filling in spaces round piles. The piles for the piers should be driven so deep that they will not sink further when the bags are stacked on the bridge The bamboos or pipes cause scour. They should not be driven in sooner than is necessary.

Art. 3. River Training for Navigation. By the "training" of a river is meant its general guidance and control, by works which are — for the most part – within its own channel though they may include works outside it. The term "training" is used in preference to "regulation" because, on canals, the latter term is often applied merely to the control of the discharge at the "regulators" or off-take works. One instance of training — that of a diversion with spurs to drive the river down it — has already been mentioned (Art. 1). The diversion may be either within the river channel or outside it. Training may include the closure of side-channels, spurs outside the channel as on the Ravi (Art. 1), bank protection and possibly some of the devices mentioned in Chapter IV., Art. 1.

The kind of training which is best known and often extends over long lengths of river channel, is that which has for its object the facilitating of navigation. In nearly all cases the main feature is a reduction in the width of the stream with the object of deepening and regularising it. The

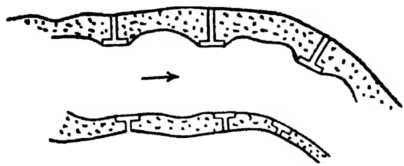
kind of river which can most easily be dealt with is one which is fairly wide and shallow. The narrowing of the stream — effected by means of training walls or spurs — deepens it, and also gives it, as far as possible, a proper course, that is, regular curves and straight reaches. As to these see Chap. VI., p. 140. The narrowing is effected sometimes from both banks and sometimes from only one. It tends to cause — at least temporarily and in the absence of other measures — a rise of the water-level and an increased depth and velocity. Such rise of water-level may be undesirable because — for instance — of floods. The rise tends to cause scour of the bed, but actual scour may or may not occur. There may be hard places which prevent general scour. At such places dredging or the removal of rock is usually carried out, either to regularise the channel or to keep down the water-level or both.

The degree to which the width of the stream is reduced varies. Wherever possible, irregularities are removed. Sometimes sharp bends are cut off by diversions. Side channels may be closed. This is equivalent to narrowing and also to regularising. In most cases the general reduction of width is not excessive. In such a case the capacity of a trained river for passing off floods may, owing to the deepening between the walls. be not less than it was before. It may, especially if some sharp bends are cut off or eased, be even greater than it was before. Sometimes the training effects a very great reduction in width, as on the Durance (Fig. 47), but this is usually in cases where the channel is soft and great scour will occur. In spite of this, however, the capacity for passing off floods may be reduced unless bends are cut off. The question of floods generally needs careful consideration when training is undertaken.

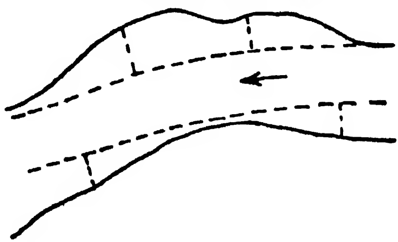
In order that a river may be suitable for training for navigation purposes the low water discharge must not be too small. Also the less the severity and frequency of the floods the better. The reaches most suitable for training are the lower ones where the discharge is less fluctuating than higher up.

In a channel trained as above, there may still be difficulty

in upstream navigation. This difficulty, if the silt carried is very heavy, may have to remain; but generally the velocity of the stream can be reduced by canalising as explained below.



A slight reduction in the width of a stream can be effected by revetting or bushing (Chap. IV., Art. 3) but a con-



siderable reduction of the width by a direct process of filling in, is generally impracticable. The expense would generally

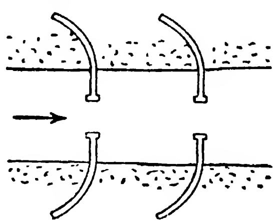


FIG 47

be prohibitive. Earth, if filled in, is liable to be washed away unless revetted all along. Reduction in the width of a large channel is nearly always effected either by spurs (Fig. 45) or by training walls (Fig. 46).

Training walls give the most regular channel and are on the whole the most satisfactory but spurs can be constructed entirely by working from the bank and are often the cheapest. They are also suitable for use on the convex bank at bends.

Usually the spurs and training walls are carried up only to a foot or a foot-and-a-half above low water-level. It is not necessary to carry them higher. Floods can thus spread out and submerge the walls and deposit silt.

Whether spurs or training walls are used, the object is to confine the stream, at least in its lower stages, to a definite zone and to silt up the spaces at the sides. The narrowing thus assumes a permanent character and becomes more or less independent of the spurs or walls.

If a rise in the flood level is not objectionable or is not likely to occur, it may be desirable to expedite the silting and to encourage it to extend up to flood level. In such a

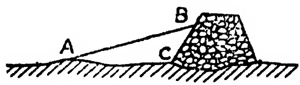
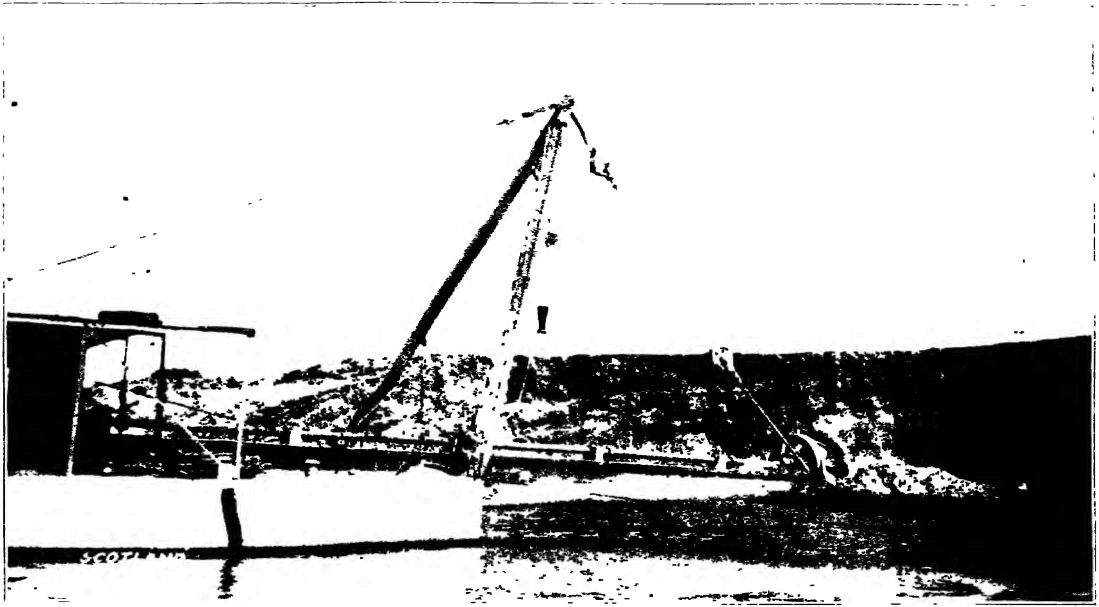


FIG 48

case in order to silt up the space between a training wall and the adjacent bank of the stream, cross walls, of the same height as the longitudinal walls, can be run at intervals (Fig. 46). The spaces when partly silted can be planted with osiers or with anything which will grow when partly submerged or subject to flooding, and this will assist the silting. If the water of the stream contains silt at all stages of the supply, gaps can be left in training walls so that silt deposit may occur at all times and not only in floods.

In a flood the lower water — below the top of the walls or spurs — flows along the trained channel, but the upper water flows more or less parallel to the natural banks of the river, so that some disturbance and cross currents occur. The floods may cause silt to deposit here and there in the trained channel or do some damage to the walls.

Sometimes in front of the longitudinal wall, spurs ABC (Fig. 48) have been placed at fairly close intervals and made



Cutter Suction Dredger

with the slope BA flat, in order to reduce the width of the stream when it is very low, and made to point slightly upstream so that water flowing over them will tend to go towards midstream [1] and cause boats to keep clear of the spurs, especially on a concave bank.

In an irregular channel the bed is irregular. The deep places are usually where the stream is narrow. When such a channel has been trained to a uniform width the surface slope in the shallow reaches (Fig. 49) may be too great. They of course tend to scour and the deep places to fill up, but this process may be slow. Spurs like those just mentioned can be constructed on both banks in deep places, increasing the surface slope there and reducing it elsewhere.

If the bed of a stream is to be lowered and is — for instance

Fig. 49.

— of hard clay, it may be necessary to dredge it and, when this has been done, over the correct width and alignment, further training may be unnecessary. If the bed is of soft mud or sand, a dredged channel is likely to fill up again, and training alone will be the method to adopt. If the bed is moderately hard, say compact sand, it may be suitable to train the channel first and then to dredge if necessary. In any case, shoals of hard material may have to be dredged or rocks, whether these form shoals or lateral obstructions, to be blasted or otherwise broken up. In cases where it is desired to train a channel and to raise the water-level without any lowering of the bed, or in any case in which the bed is likely to scour to a lower level than is desired, or if the bed is to be raised, the submerged walls or weirs described in Chap. IV., Art. 1, may be adopted.

Dredgers. — Dredgers can remove mud, sand, clay, boulders, or broken pieces of rock. The "bucket ladder" dredger is the

[1] *Ency. Britannica*, 11th. Edition, Vol. 23.

commonest type. The "dipper" dredger is another. Both these can work in depths of water ranging up to 35 feet. The "grab bucket" dredger can work up to any depth and in a confined space. With the dipper and grab bucket dredgers the actual work done under water is intermittent. The "suction dredger" pumps up mud or sand mixed with water. A dredger may be fitted with a hopper or movable bottom, by means of which it can discharge the dredged material — this, however, involves cessation of work while the dredger makes a journey to the place where the material is to be deposited — or it can discharge into hopper barges or directly on to the shore by means of long shoots. For small works in comparatively shallow water the "bag and spoon" dredger, worked by two men, can be used. Bucket dredgers have been largely superseded by suction dredgers. Clay and other material can be dealt with by the suction-cutter or drag-suction types.

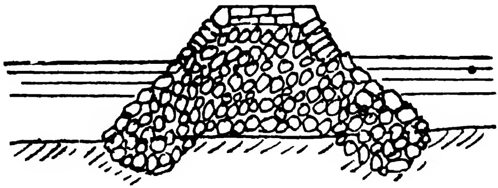
When rock has to be removed under water it is blasted or broken up by the blows of heavy rams provided with steel-pointed cutters. The broken rock is brought up by dredgers. A combined dipper dredge and rock-cutter is in use and enables much time to be saved.

The Twante Canal in Burma, constructed to shorten the distance between the Rangoon river and the China-Ba-Kir river, was made by dredging, the liquid spoil being deposited between a bank on the edge of the channel and another bank further away.

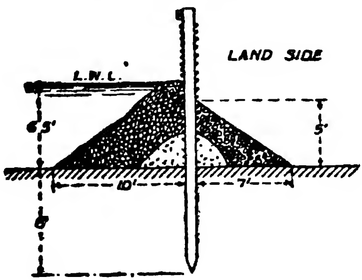
The cost of dredging varies greatly with the nature of the work and the locality. It has recently been shown by Berridge that, in the case of a ladder dredger, the actual horsepower expended in dredging bears to the horsepower calculated merely from the weight of material raised, a ratio which is 3.5 for mud, 5.1 for ordinary clay, 6.8 for clay with stones and boulders, and 7.8 for hard compact red sand. Much work is done in dragging the bucket along the bottom and in excavating the material. For other kinds of dredgers the ratio is believed to be approximately the same. [1]

[1] *Proc. Inst. C.E.*, Vol. CC, p. 421.

In widening a channel — for instance where a corner is cut off — the excavation can be carried down in the ordinary way to below the water-level, a narrow piece of earth, like a wall, being left to keep the water out. If the channel cannot be laid dry, the work can be finished by dredging.



Training Walls and Spurs (Details). — Training walls are commonly made of loose stone (Fig. 50) dropped from barges. Above low water level the stones may be placed more carefully by hand. When scour of the bed is expected, two parallel trenches are first dredged, as shown in the figure, to below the depth of probable sccur. Such a wall is generally the most satisfactory kind. The side-slopes may be about $1\frac{1}{2}$ to 1. If the channel is one which occasionally runs dry

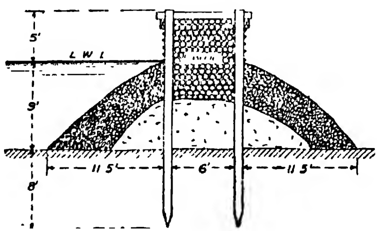


or nearly so, the whole of the facing can be laid by hand and the wall made thinner, the top width being reduced. Other kinds of wall are used where stone is expensive or scarce, and timber is not expensive.

In some cases — as on the Garonne — the walls have been made of rows of wattled piles (Fig. 51) about 2 feet apart

and reaching up to high water level, the tops being connected by waling pieces and the foot protected by gravel and this again covered with shingle or rubble. Floods rose 20 feet and completely submerged the walls. There is less gravel on the land side because silting is expected there. On the river side some may be washed away. Sometimes there are two rows of piles (Fig. 52) the intermediate space being filled with fascines. Timber is of course perishable. The cases most suitable for its use are those where rapid silting is probable so that dependence on the walls will soon cease.

Spurs for bank protection are described above (Chap. IV., Arts. 1 and 3) and also the swirls which they may cause



and the remedies to be adopted. Spurs for narrowing streams are usually longer than those used for bank protection. They are generally at right angles to the stream. They are best suited to a hard channel such as one of gravel or shingle. When they have to be long their distance apart is made greater. As already stated the tops are usually not much above low water level. The material used is commonly loose stone. In other cases spurs have been made very much as in the case of the training wall shown in Fig. 52, but with stone instead of gravel or shingle. Trees weighted with stone in nets, trees alone, mattresses or lines of stakes can also be used as in spurs for bank protection. In some large rivers "hurdle dykes" mentioned below, are used.

Spurs are also made of earth — generally faced with gravel or pitched with stone — the heads, which are generally T-

heads (Fig. 53) being of large stone or of earth pitched with stone. Such spurs are carried up to above high water level, allowance being made for any rise of such level due to the narrowing of the stream. When the spur is very long most of the shank is usually of earth only. If the land near the stream is liable to be flooded, an earthen bank — in continuation of the spur — may be made across it so that the spur will not become an island with a possibility of a channel being formed between the spur and the bank (Fig. 47).

Training work on the lower reaches of the Mississippi is different from that on most rivers. The stream is of exceptional size, the width being 4,000 feet or more. The works, carried on for a great number of years, have had for their

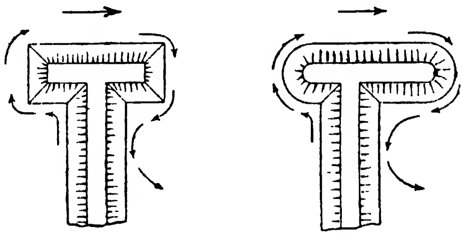


FIG 53

object the formation of a channel about 3,500 feet wide with a low water depth of about 10 feet. Spurs have usually been adopted and they have been "hurdle dykes" or else of brushwood weighted with stone. A hurdle dyke consists of one to four parallel rows of long piles spaced 8 or 10 feet apart, the distances between the rows being about 20 feet. The piles are tied together — in both directions — by horizontal timbers and in some cases the upper row of piles has brushwood woven into it Such a spur is permeable, the water passing through it, and this facilitates the deposit of silt downstream of the spur. Much work was also done by dredgers which cut through the shallows or "bars".

On the middle Mississippi — between the mouths of the Missouri and of the Ohio River — the object of the training is to maintain a navigable channel 200 feet wide

with a depth of water of 8 feet to 6 feet. The spurs are hurdle dykes. Formerly dykes were used made of stone resting on mattresses. These were found difficult to maintain.

For further details of training walls and spurs see Art. 4.

Alignment of the Trained Channel.—The alignment of training walls or spurs should be such as will give the best channel consistent with economy in cost. The best channel is generally that which is most free from irregularities (Fig. 54). A certain amount of choice of alignment is always afforded by the reduced width of the trained channel and by small diversions or easings of bends. Where a curved reach ends — the curve being fairly sharp — and the next reach is

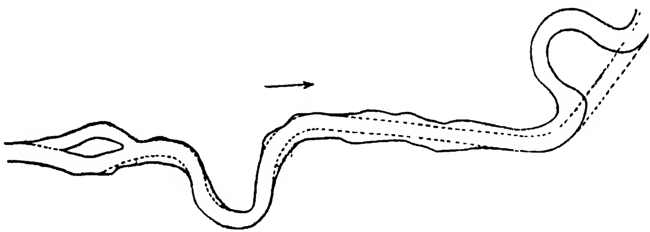


FIG 54

curved in the opposite direction, a short straight reach should be interposed. This is desirable to allow room for the bed to alter its form, the deep portion having to cross over (Chap. III., Art. 3). The width between the training walls should generally be the same throughout, whether the reaches are straight or curved.

It is sometimes said that straight reaches are objectionable because the stream will tend to wander from side to side and cause shoals, whereas in a bend there will be no such tendency. Any such tendency will be greatest at low water, but it is likely to be serious only when the width between the training walls is too great. If the width cannot be made such as to do away with the trouble — as for instance where the stream is liable to periods of very low water — the spurs with flat slopes, described above, should be constructed in front of the walls. Otherwise it may be

suitable to adopt a curved course, adding the spurs only in the short straight reaches where the direction of the curvature changes.

At a bend which is fairly regular the concave bank — or part of it — should form one wall. It can — after slight adjustments as to alignment — be protected by revetment. The convex bank can be dealt with by a series of spurs. This arrangement was adopted on the Rhone [1] in some reaches.

If the bend is regular but very sharp the stream may be so narrow that nothing need be done to the convex bank.

The plan of revetting one bank can sometimes be adopted when it is straight. In a reach in which the river is roughly straight but irregular, the trained channel can be straight (Fig. 54). This is especially suitable if the river is everywhere shallow. But if it is deep near one bank or the other, the trained channel can be made to cross over one or more times in such a reach In such a channel the deep portions cannot be located *a priori* (Chap. III., Art. 3). It would hardly be profitable to increase the number of crossings over, in order to obtain a series of sharp curves needing no work on their concave banks Each concave wall would have to overlap the next one. There would be an increased liability to damage by floods.

If, instead of on the concave banks revetments were made on the convex banks. the length of the trained channel would be reduced but the deepest parts of the channel would be abandoned and this is generally a more important consideration. It is true that a succession of sharp bends is equivalent to an increase in roughness in a channel but instead of easing all bends by crossing to the convex banks, it is preferable to construct a diversion here and there.

General Remarks. — The steps so far described, exhaust the list of what can be done so long as only the cross-section of a stream is dealt with. But a change of gradient may be required. The gradient can be steepened by means of diversions or flattened by introducing weirs. When the scheme

[1] Ency. Britannica, 11th Edition, vol 23

includes weirs and locks it is known as "canalising".[1]

Suppose that, in the case of an alteration like that shown in Fig. 55, the mean depth is doubled. The new width will be about equal to $\frac{1}{2.8}$ of the old width. If this gives too narrow a channel it may be desirable to flatten the gradient. If it gives too wide a channel a greater depth can be adopted. While the width and depth of the stream will be fixed so as to be suitable for the navigation, the ratio of depth to velocity can be so arranged, if this is possible, as to minimise trouble connected with silting or scour (Chap. VI, Art. 6).

The weak point in a scheme which includes weirs is the difficulty of dealing with floods. A scheme perfect in all other respects may be vitiated because of the obstruction caused by weirs to the passage of floods. The difficulty is got over by means of sluices or "movable weirs". The subject of weirs is dealt with in Chap. VIII.

FIG. 55

When the work on any reach of a channel includes the raising of the water-level, towards the lower end of the reach, by a weir or by narrowing the channel — though in the latter case the raising may not be permanent — it is generally best to commence the work from the upstream end The raising of the water-level will not then interfere with the execution of the rest of the work In any case in which there is doubt whether the whole of the scheme will be carried out, the reach to be dealt with first can be decided on according to circumstances. There is no general reason for selecting an upstream or downstream reach, except that any raising or lowering of the water-level will extend upstream of the reach and not downstream of it.

Art. 4. Other River Training Works. Training, similar to that described in Art. 3, that is depending mainly on re-

[1] See also p 177 Diversions and weirs may be made together Diversions may be merely for shortening the channel and getting rid of sharp bends

duction of width and extending over considerable lengths of river, may be adopted in order to prevent or reduce the deposit of silt in canals.

The head reach of the Wali Mohamad irrigation canal in the Punjab was a creek — of the river Chenab — which had been frequently cleared out and had become a part of the canal. Its width was about 100 feet. The width of the canal downstream of the head reach was 80 feet. The head reach silted considerably and it was therefore narrowed, the bottom width being made 75 feet. The narrowing was effected by means of lines of stakes and bushing (Chap. IV, Art. 3) on both sides of the cleared channel (Fig. 56) the bushing extending up to 5 feet above the cleared bed, which is shown by dotted lines. Cross lines of stakes and brushwood — upper dotted lines — were added. The level of the cleared bed ave-

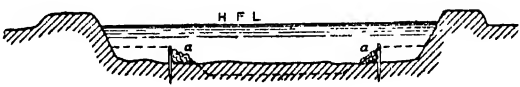


FIG. 56.

raged some 1.5 feet below the silted bed. The depth of water on the cleared bed was 3 feet when the canal was opened, and rose to some 10 feet during the highest flood, the regulator, where the supply is controlled, being some miles below the head reach. The banks were steep and fairly hard so that a single line of stakes sufficed wherever one bank ran straight or in a regular curve.

The water being highly silt-laden, heavy deposits occurred on the outside of the lines of stakes while deposit in the cleared channel was much reduced. While the water was rising, the bushing and stakes suffered damage and required constant attention and repair until the floods put a stop to work. In the following season the work to be done was much less. Whether training work of this kind is worth attempting — at least in India — depends largely on who is to look after it. With care and attention it may succeed admirably. Otherwise it will probably fail.

On the Rayah Behera and Ibrahimiah canals in Egypt

the banks used to fall in and the channels to widen and silt. This was remedied by systems of spurs. The Rayah Behera was 65 feet wide and the banks were of sand. The spurs were generally about 650 feet apart in the straight reaches — nearer together on curves — and were made of bricks laid dry, the ends being sloping as in Fig. 23 (p. 89) but at 1 to 1. The depth of water in floods was some 20 feet. The Ibrahimiah canal was 200 feet wide and in stiffer soil. The depth of water in floods was some 30 feet. The spurs made were of rubble stone sloping down at 5 to 1 the ends being at 1 to 1. They were built in pairs opposite to one another and at intervals of about 800 feet. All the spurs were above flood level at their landward ends [1].

It has been seen (Art. 3) that a system of training works may extend over a long length of channel and to both banks of the stream Also that locally it may consist of bank protection or a diversion — perhaps with spurs designed to assist its working — or with the closure of a branch. Such local works are frequently constructed — without any connection with long training works — the object being to shift the stream into a more convenient position. If it is threatening some bridge or weir or a building near the stream, it may be easier and safer to shift the stream than to be constantly adding to bank protection. Again it may be necessary to keep the stream flowing close to one bank at wharves or at the off-take of a canal which has to be kept supplied. The procedure and methods to be adopted in these cases do not differ from those already described. Some such works are dealt with in Chap. VIII., Art. 6

[1] *Irrigation Works in India and Egypt.* Buckley.

Notes to Chapter V.

Closures of Flowing Streams (p. 107). In a river in Canada the stream was in the right-hand side of the channel. In the left-hand part, which was dry, a weir was built with suitable foundations and protection and with piers and gates. Close to the stream there was built a large block of concrete with height equal to the width of the stream and so shaped that when it fell into the stream it would closely fit the channel. Its base was blown up so that it fell in and stopped the flow of the stream, which then headed up and passed over the weir.

The Islam weir on the Sutlej (Chap. x. Art. 1) was constructed on dry land in a bend of the river. After its construction the river—discharge 2,500 c. ft. per second—had to be diverted and made to pass over the weir. Earthen embankments were made on both banks and continued into the stream until the earth began to be washed away. In the gap—some 500 ft. wide—a mattress was laid down. It reached from 38 ft. upstream of the centre line of the embankment to 82 ft. downstream, and was laid down in 20-foot strips. Two pieces, each 60 ft. by 20 ft., joined end to end and wired together, formed a strip. On the mattress stone was thrown until it rose above the water—the water was, of course, headed up—and the whole discharge passed through the stones. Leading cuts which had been dug along the new course were opened and the stone embankment was tamped on its upstream face. A freshet in the river caused a breach in one of the earthen embankments, but it was closed.[1]

River Training for Navigation. If the navigation of a river is impeded by rapids, it is often suitable to construct a weir and lock (p. 176) to replace one or more rapids, as in the case of the Kachlet rapids on the Danube.

The upper Mississippi, from Minneapolis to the mouth of the Missouri, is 664 miles long, and at the latter point the discharge ranges from 10,000 to 375,000 c. ft. per second.

[1] *Selected Engineering Papers*, No 69. Inst. of Civ. Engineers, 1929 (Downing).

The river is often split up into two or more channels. The banks are fairly permanent, changes in the channel gradual and the water clear. The training aims at securing a depth of 6 ft. The narrowing is effected by "wing dams"—spurs—of brush and stone in alternate layers, running out nearly at right angles to the bank. At bends the spurs, if any, are on the convex bank. In all but low stages of the river they are submerged. They are liable to damage from floating ice.

In the Middle River (p. 123), from the Missouri to the Ohio—200 miles—the bed and banks are easily scoured, and there is much silt, drift and ice. The depth aimed at is 6 ft. to 8 ft. A dyke is constructed on a woven brush mat, 125 ft. wide and the full length of the dyke. One to four—generally three—rows of piles are driven through the mat. The piles in any row are 8 ft. to 10 ft. apart and the outside rows are 9 ft. apart. Each pile in the centre row is tied to the two piles nearest it in the outer rows. Dredging is carried on to some extent. Floating rubbish collects against the dyke and is weighted with stone till it reaches the mat. After this a single flood often silts up the space between consecutive dykes, and willows grow on it, giving a permanent bank 20 ft. above low water level.

The Lower Mississippi (p. 123) is 2,000 miles in length, and the discharge at Columbus ranges from 71,000 to 2,015,000 c. ft. per second. The valley is wide and the "bluffs" are far from the river. The banks are high and easily scoured. The navigable channel aimed at is 9 ft. deep and 250 ft. wide. Training would be too expensive. Bank protection alone is carried on, except where a bar has to be dredged. This is less costly and more certain than training. The revetment mattresses may be 400 ft. wide by 1,000 ft. long, and are made of fascines of very heavy brush, 16 in. diam., woven together by ¼-in. galvanised strands, which are clipped at 10-ft. intervals to ½-in. cables. "Framed mattresses" of three layers of willows—the middle layer at right angles to the others—pressed between timber frames, are also used.[1]

[1] *Engineering News Record*. Vol. 94, p 508, and Vol. 104, p. 720.

CHAPTER VI

WORKS FOR DRAWING OFF AND UTILISING WATER

Art. 1. Artificial Channels. This chapter deals with open artificial channels and with the works in which they are most used. The great majority of open channels are canals in earth. The longest are used for irrigation or navigation. Others are used for hydro-electric, drainage [1] and other works. Short channels, or parts of long channels, are frequently made of — or lined with — wood, plaster, concrete, brickwork or metal.

A canal is supplied either from a river or from a reservoir in which rain water has been collected. Reservoirs may also be used as supplementary sources of supply or for storing water when it is in excess of requirements and giving it out again at other times. Sometimes a supply is drawn partly from one river and partly from another.

Before a long channel — or a system which includes channels — is designed in detail, preliminary investigations are carried out and rough surveys made. These preliminaries may cover various alternative schemes. They are enormously facilitated if accurate maps of the country are available. If the maps are contour maps, preliminary surveys are probably unnecessary. From a contour map, the features and lie of the country can be easily seen and approximate profiles of any proposed line can be prepared. An extensive irrigation system cannot be properly designed without a contour map. In other cases it may suffice to run some trial lines, but if the line is long and there is much choice as to the alignment it is, in the absence of contours, almost impossible to say that the line selected is the best possible.

[1] For drainage works see Chap. VII, Art. 5.

It is necessary to collect very complete information as to the supplies of water which will be available and the seasons at which they will be available (see also Chap. I, Art 2). Until this has beeen done it is in most cases impossible to properly design a system which depends on the utilisation of a given quantity of water. When the supply is obtained from a river the nature and quantities of the silt and solids transported — when they are considerable — must also be carefully studied

The preliminary investigations should include borings, in order to ascertain the nature of the soil in the tract to be traversed by the canal, down to the level to which excavation is likely to be carried This knowledge may prevent time being wasted in designing channels along lines where the soil is unsuitable Porous material such as gravel and sand should be avoided if possible. Rock, though forming an excellent channel, involves great expenditure in excavation. Peat is porous and unsuitable Sand, though easy to excavate, involves flat side slopes and possibly their protection or covering them and the banks with good soil. A bank of sand may be worn down by winds or by cattle and much of the sand — especially in the former case — goes into the canal Soil impregnated with salts is most unsuitable for a canal. As to sand see also p.

Streams and drainages which cross the proposed alignment must all be closely investigated and their maximum discharges — probably occurring only on rare occasions — estimated (Chap. VII). Generally they have to be taken across the canal by syphons or aqueducts. If any heavy drainages are crossed it may be best to alter the alignment of the canal if this can be done and if they can thus be avoided. If small — even when in flood — the stream can perhaps be diverted or, if fairly free from silt or if settling tanks can be constructed, allowed to enter the canal. Occasionally, among hills and if they are at no time silt-laden, they can be used to give a supply to the canal during rainy weather. At such thimes the usual head of the canal may have to be temporarily closed because of heavy silt in the

river. One stream to be crossed may possibly be the river itself. A high cliff may come down close to the river. Instead of taking the canal through or under the cliff it may be preferable to take it across the river, or to shift the river away from the cliff.

In places on the proposed line of canal there may be steep side-long ground. The strata — especially when cut into — may be liable to slip. The liability is greatest when there is much loose material or when the strata are thin and numerous or when hard and soft strata alternate. It may be increased by the percolation into the ground of water from the canal. Sometimes tunnels are used. They may shorten the line or otherwise be economical. If a tunnel — or even its upper part alone — is confined to hard rock, lining will not be necessary though water may give trouble. The geological features of the locality should be studied and experts called in if necessary.

The head reach of any canal taking off from a river which is apt to shift its course, or to cut deeply into its bank, should not run parallel to and near to the river bank but should be aligned so as to get well away from it Nor should any canal at any part of its course come near to such a river

Regarding the need for keeping clear of country which is merely subject to flooding, see Chap. VII., Art. 3.

Regarding losses of water from earthen channels such information as is available is given in Art. 2. A long canal is nearly always unlined except in special places. The question of the losses of water is now, however, receiving increased attention. Puddle has always been used, more or less, on navigation canals (Art 5) Some irrigation channels in the United States are lined with concrete or plaster Earth suitable for puddle is often unobtainable. Suggestions have been made as to lining some large projected irrigation canals in India with concrete. Such lining enables a higher velocity to be adopted, the size of the channel to be reduced and perhaps "falls" omitted, but the cost will probably be in excess of that of an earthen unlined channel. The cost of

maintenance will, however, be less. Much depends on whether the supply of water is in excess of the actual requirements. If water has to be collected in expensive reservoirs this is a good reason for using lined channels. In irregular ground, channels of concrete and other structural materials may be cheaper than channels in earth.

The question of future extensions of the system should also be considered. The question of lining may have a direct bearing on it. It may be possible to obtain the water for an extension by reducing the losses, though the addition of lining material necessitates, in most cases, a long closure. Again the question of lining — or of the use of structural materials — is, as has just been seen, connected with that of irregular ground.

Questions such as those mentioned in the preceeding paragraphs should — in cases where they assume importance — be dealt with in the early stages of a project. They may involve alternative schemes differing largely from one another.

The discharge of the canal — perhaps only roughly fixed at the outset — must at an early stage be decided upon within narrower limits. The methods by which it is calculated differ according to the class of canal and are explained in Arts. 3, 4 and 5.

The discharge of a canal fluctuates from time to time as will be seen, but in all cases the channel has to be designed to carry a certain maximum discharge or "full supply", whether or not such supply is always — or usually — carried and whether it is all utilised or some at times passed into escape channels or otherwise run to waste. The banks have to be made to hold up the water to a given "full supply level".

Off-Takes [1]. — The proper locality for the off-take of an artificial channel from a river depends on the "objective" or point at which the water is to be delivered. In an irrigation work this is the highest portion of the area to be irrigated. In a work for hydro-electric power it is the "forebay" whence

[1] Also called "Intakes".

the water is sent down to the turbines. It may be a reservoir.

The proposed discharge of the channel and the water-level at the objective point having been determined, the channel is designed so as to run back from the objective with its water-level at a suitable gradient, and to meet the river where the water-level of the latter is high enough. If a weir is to be made in the river this is of course taken into account.

The above considerations determine the approximate position of the off-take. Its precise position depends on the river conditions. The height of the river bank should not be so great as to give rise to needless expense in excavating, but it should if possible be such that the banks of the canal will be above flood level without much extra expense being incurred. If the stream is one which does not appreciably shift its course or alter its channel almost any site may be suitable when the stream flows close to the bank and the latter is of suitable height

If the river has a tendency to erode one bank and to silt at the other but such tendency can be controlled at a reasonable cost, it may be best to select a site on a curve which is gentle and where the bank is concave to the stream. The bank can be protected by any suitable method and there will be no danger of the stream moving away from the off-take or depositing a silt bank close to it. Possibly some training works may be necessary, but if by going some distance up or down the river a site can be found where no such works will be needed it will probably be worth while to shift the site of the off-take accordingly. See also p

The "Headworks" of the new channel include, especially in the cases of many irrigation canals and power canals, a weir across the river. The designs of weirs and of headworks are dealt with in Chap. VIII. In some cases no headworks are necessary, as for instance where surplus water entering the channel can be got rid of without trouble.

Any advantage in the matter of water-level, accruing to

the canal owing to the direction of the off-take or the curvature of the river is not generally important. When there is a bend in the river, a canal taking off on the concave bank has an appreciable advantage over one on the opposite bank only in the case of a sharp bend or a high velocity, such as occurs, for instance, in floods or in a rapid stream flowing in a rocky channel (Fig. 57). The discharge of the canal is increased by a spur just below it, and decreased by one just above it. On some irrigation canals in India, where the velocity is high and the channel of boulders, the

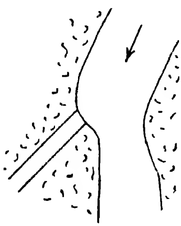


Fig. 57

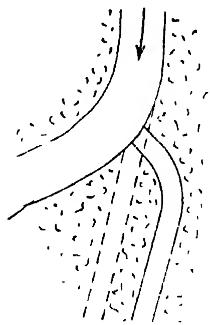


Fig. 58

farmers sometimes run out small spurs below their water-course heads in order to obtain more water

It was once not unusual in India to build the heads of irrigation distributaries at an angle of 45° with the canal, but they are now built at right angles as shewn in Fig. 16 (Chap. IV, Art 2, p. 78) The velocity in the canal is 2 or 3 feet per second, and that in the distributary less. The water surface nearly always has a fall into the distributary head. It is not often that the head is fully opened and that the distributary has to take all the water it can get. For any channel in which this condition does often occur the advantage of a skew head may be appreciable if the velocity is high. In such a case the head can be built skew. It will be more expensive. See lower part of Fig. 17 (Chap. IV, Art. 2, p. 79).

If an off-taking channel is skew to the bank (see dotted lines in Fig. 58) and the head is built square to the channel in the usual way, silt — if the river carries it — will accumulate in the triangular space above the head when the channel is closed. It is much better to make the off-take at right angles as shown by the firm lines. The small curve can be accepted as will be seen below.

If the regulating head is placed, not at the actual off-take but a short distance down the channel, any checking of the supply causes a reduction in the velocity — if not still water — in the "pocket" between the off-take and the work. This, if the water transports silt, causes deposit in the pocket. Such an arrangement is adopted sometimes in the case of drains where they join the English fenland rivers. When the tide rises, the gates close (Chap VII., Art. 5) and silt deposits. It has to be cleared out occasionally. The embankments have been made with flat slopes on the river side so that the roadway on the top (where the regulating bridge is situated) is some distance from the river channel In Western India the heads of some irrigation canals or their branches have been set back in the same manner. Generally this arrangement would not be approved of.

It may sometimes be convenient to place the regulating work a long way from the off-take In India this is necessary in the case of irrigation canals taking off from shifting rivers. Otherwise any erosion of the bank would destroy the works The supply entering the canal is unrestricted At the regulator — perhaps miles from the off-take an escape is usually provided

At an off-take in ordinary soil the corners become rounded as in Fig 6 (Chap. III, Art. 4) If any attempt is made to gain in velocity of approach by means of a sharp angle at F, it is quickly washed away and a silt bank forms at DG the off-take becoming very much as shewn It is stated by Buckley [1] that, in the case of canals in Egypt, the water in the canal tends to rebound from side to side as it goes down the channel and erodes the bank and that it is not

[1] *Irrigation Works in India and Egypt*, Chap IV

unusual to revet the channel (Fig. 59). This does not occur in the very numerous Punjab canals, where the velocities are higher.

The canals in Egypt are also taken off where the main stream of the river is close to a concave bank, a position carefully avoided on the Punjab rivers. The tendency to heavy erosion of the bank in such reaches has been described (Chap. III., Arts. 3 and 4) and when it occurs the water on that side of the channel is heavily charged with silt, especially towards the lower end of the reach, and heavy silting occurs in any channel having its off-take in the reach.

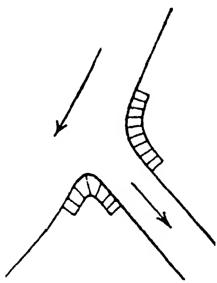


FIG. 59.

Details as to the difficulties attending irrigation canal off-takes in great shifting rivers are given elsewhere [1]. Works for training the rivers are quite out of the question.

The question of silt deposit in canal head reaches is dealt with in Chap. IV. Sand traps are considered in Art. 1 of that chapter and shown in Figs 11 and 12, p. 74. Sometimes the bottom is given a steep transverse slope towards the scouring gate or gates These should be large enough to enable the canal to be quickly emptied. A sand trap may be necessary close to the off-take of a canal. It can be constructed there just as easily as elsewhere, provided that there is sufficient head and that there is water to spare for scouring. Another plan is to construct a sluice-way through the weir

[1] A further work under preparation

in the river, close to the off-take of the canal but at a lower level. Stones and sand can be passed through it without entering the canal.

Ice. — In Canada and other countries where the winters are cold, much trouble may be caused by ice. Besides surface ice there is "anchor" ice, which forms in crystals on the beds of rivers and adheres to the bed. Its formation is due to radiation from the bed through clear water and ceases, or is reduced, when the surface becomes frozen. There is also "frazil" ice which consists of crystals floating in the stream. It may be formed in such quantities that the stream becomes a viscous mixture, the water rises and it may overflow the banks. When in this condition the river may freeze over. The water then flows under the surface ice and the formation of anchor ice and frazil ice ceases. In the spring when the ice on rivers is breaking up there may be great quantities of floating ice and "ice-jams" may occur. To deal with ice local knowledge and experience are required.

At the site of a proposed off-take in a cold country it is desirable to study the prevailing winds and to design the off-take so that ice shall not be driven into the canal. The position of submerged rocks should be ascertained. They may become coated with ice and affect the supply of the canal. If there is a weir or dam below the off-take and if the water is frozen over, it may be an advantage because anchor ice and frazil ice will not be formed.

General Principles of Design. — In designing a canal due weight is naturally given to the cost of maintenance. A small initial cost may involve high maintenance charges. It may however be unavoidable. If so, it is desirable to study methods by which improvements — such as will tend to reduce maintenance costs — can be effected later.

The discharge having been determined, the next question is that of the velocity The channel should if possible be designed so as neither to silt nor to scour The general laws affecting these matters have been stated in Chap. III, Arts. 2 and 3. A velocity which suits the bed of a channel may not suit the banks and vice versa. Further details are given

in Art. 6 of this chapter. It may be that owingt of latness of the gradient, the velocity of the stream cannot be high enough to cause scour and that no great quantity of silt will be brought into it [1]. In such a case it is desirable to adopt as direct a course as possible so as to obtain the highest possible velocity and the minimum cross-section. This tends to minimise both the cost and the loss of water.

The most economical bed level is that which gives the "balancing depth" of excavation, making the quantity of earthwork in the banks the same as in the excavated channel. In this case none of the excavated earth is wasted, or has to be led to a distance, and no more has to be got, or brought from a distance to make the banks The full supply level in such a canal is generally above the ground level. Such a section can only be attained - over a considerable length of channel — when the ground slope is parallel to the bed gradient of the channel, but it can often be approximated to.

The general alignment, bed level and gradient having been thus provisionally determined the line should be marked out with any adjustments which may be needed in order to avoid the acquisition of expensive land, or the subdivision of land by cutting across it, and also to avoid particular places where the depth of digging or the height of the banks will obviously be excessive. The existence of such places is not always known even from a contour map. An embankment across low ground is not only expensive but is liable to breach Borings or trial pits, if not made before, are at this stage necessary all along the line. They may indicate further deviations. Levels having been taken over the line finally selected, a longitudinal section can be prepared.

If the velocity of the water in the proposed canal is otherwise too high it can be reduced by introducing falls. These are placed if possible at points where there is a steep drop in the ground level so that the general depth of excavation remains as nearly constant as possible. Or it may be suitable

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[1] In the case of a navigation canal — not being a canalised river — the velocity is negligible and no appreciable quantity of silt can be admitted.

to give the canal a more circuitous route and so to flatten the gradient.

In canals for power, falls are seldom — if ever — introduced until the power house is reached because a great object is to avoid loss of head. When falls are used in any other class of canal it should be considered whether they are likely to be ever used for power. If so the number of falls should be reduced as far as possible and their individual heights increased.

If trouble is likely to occur from silt deposit and not from scour, the canal can be given the steepest possible gradient and a suitable ratio of D to V, all the steps possible being also taken as to the exclusion — or scouring out — of silt from the canal. Silt deposit generally occurs first near the off-take. This may be the most troublesome place for it because it at once reduces the discharge unless the water-level at the off-take can be raised. In a long canal the gradient in the head reach can be steeper than elsewhere.

If the channel is a long one it should be divided into reaches, each of which can be designed separately.

Bends have been considered in Chap. III., Art. 3 and — as to loss of head — in *Hydraulics* (Chap. II., Art. 13 and Chap. VII., Art. 1). If they have in any reach to be numerous and sharp, their effect amounts to an increase in roughness of the channel and can be allowed for. The total heading up due to a curve of 90° is not exactly known. At the most it is perhaps $.50 \frac{V^2}{2g}$.

If the radius of curvature of the centre line of a stream at a bend is too small relatively to the width (W) of the stream there will be eddying and disturbance, a silt bank being probably formed at the convex bank and erosion taking place at the concave bank. The chief trouble caused by bends is the falling in of the concave bank. A minimum suitable radius of curvature in ordinary soil when V is 3 feet per second, is 6 W. This may be halved if V is 2 feet and doubled if V is 4.5 feet per second. With such radii

the concave bank will need protection. In very rapid streams the concave bank — and in some cases part of the bed — is protected by pitching. This was done in the Upper Jhelum Canal, Punjab, where V is some 4 feet per second and the bed width 215 feet, D being 9.6 feet and the radius of the bend 2,000 feet. Where possible a radius of 5,000 feet was given for such widths.

Where the channel has in any case to be strongly protected, as below a regulator or fall, there is no objection to a curve being sharper than the limit above stated — on inundation canals it sometimes is so — or even to there being an elbow in the alignment. Below a vertical fall the water has in any case to start off afresh.

Banks. — The parts of a canal which it is most difficult to get well made are not generally masonry works or other special structures, but banks. Good banks are of the greatest importance especially where they are high. It is true that a bank which has water constantly against it and is of sufficient dimensions, nearly always becomes almost water-tight in time, but the time is less or greater according as the soil is better, and according to the amount of care with which the bank is made

In good earthwork the earth is or should be deposited in layers but the earthwork generally contains, to a greater or less degree, clods and hollows. An earthen bank which is to hold water should be made with special care, the layers being thin and clods being broken up. To make it specially good each layer should be moistened and rammed. Before the bank is made the ground under it — unless it is sandy — should be ploughed. Most soils make good banks with proper care in construction

When a bank is of great height — or wherever it appears to be necessary — additional steps can be taken in constructing it, just as in the case of a flood embankment (Chap. VII., Art 4). See also p 75.

It is a great advantage to have a road running along the top of a bank. The traffic — of whatever description — effectually consolidates the earthwork and keeps away

burrowing animals. Breaches in such banks are not likely to occur.

Art. 2. Losses of water. It is convenient to consider in this article the losses from all large bodies of water in contact with earth, that is the losses from reservoirs as well as from channels. The laws governing the losses are imperfectly known. The most important factor is probably the nature of the soil and subsoil. Others are the age of the reservoir or channel and the quantity of silt in the water. Silt even if very small in quantity constantly tends to render the bed and sides water-tight. The information available is often only of a general character, details as to soil for instance, not being available.

In a high embankment with narrow banks, absorption ceases when the water reaches the outer slopes, except in so far as it is evaporated from the slopes. If banks of sand are constructed on a layer of clay (Fig 60) and well rammed,

FIG. 60.

the absorption ceases as soon as the banks are saturated and the channel then holds water as well as any other except for evaporation from the outer slopes, but if the bed and subsoil are also of sand the absorption of the water is far greater. If a bottle is filled with water and a small sponge jammed into the neck and the bottle turned upside down, the sponge becomes saturated but no water is given out. But if a dry sponge is placed in contact with the wet one it absorbs moisture until saturated. See also Chap. II., Art. 4.

When water is turned into a dry reservoir or channel or a field, the loss is at first great. It decreases hourly and daily, and eventually becomes nearly constant, tending to reach a fixed amount Observations made by Kennedy on loamy fields near the Upper Bari Doab Canal in India showed that on a field previously dry, the rate of absorption is given by the equation,

$$y = .0891\ x^{.86}$$

where y is the depth of water absorbed in feet and x is the

time in hours. The observations extended over eight days. Denoting by c the depth of water in feet absorbed in one hour, it was found that on a field on which no rain had fallen for two months, c was .04 to .05, but on the second watering of the crop about a month later c was .02 to .03 and about the same on a third watering. It was found that at first the rate of absorption was much affected by the state of the surface of the ground but that the effect was only temporary. The losses were found to be as follows:

Day	Loss per Day (feet)	Average loss per Hour (c) (feet)
1st	1.36	057
2nd	1.13	.047
3rd	1.07	.046
4th	1 02	.043
5th	96	.041
6th	.90	.037
7th	80	.033
8th	.77	.032
	Total 8 03	

In the eight days the total loss was almost exactly eight feet.

The loss from a reservoir is reckoned in depth per year over the area of the water surface This is nearly the same as the wet area. The loss from a channel is reckoned in depth per day — or per hour — over the wet area or in cubic feet per second per million square feet of wet area. One foot per day is the same as 11.57 cubic feet per second per million square feet.

From a reservoir the loss from percolation and absorption is generally small except perhaps in the first few years. Reservoirs are generally in ordinary rolling country or in moorland. The silt deposit, even if small, tends to make the bed water-tight. The loss is generally far less than that from evaporation and, in the British Indes, is usually neglect-

ed. In Indian reservoirs the loss from percolation is generally very small. That from absorption is generally less than half that from evaporation. The whole loss from evaporation, percolation and absorption is 3 to 10 feet per year [1]. In the Bombay Presidency the total average loss is often taken to be 4 feet [2] but is probably more. The evaporation alone is probably 4 feet (Chap. II., Art. 3). In Rajputana where the soil is sandy the total loss has been found to be .20 in. to .56 in. per day — say 11.7 feet in a year — and in Madras .13 in. (December) to .33 in. (July) or about 7.4 feet in a year. In the United States and Germany 2 to 3 feet are usually allowed, in Australia and South Africa generally about 6 feet, in the hot and arid tracts of the United States and Australia 8 to 10 feet. The above figures are liable to be exceeded in particular instances. Several cases are mentioned by Davis and Wilson [3] in which the losses from reservoirs in America were excessive — generally so great as to render them useless — owing to the underlying rock being badly fissured, or of a kind — gypsum — easily eroded and partly soluble, or sandstone containing seams, or lava containing seams and crevices In another case a badly leaking reservoir recovered in the course of 8 years, the subsoil having gradually become filled with water which could not escape.

In earthen channels the losses of water from percolation and absorption are generally spoken of as losses from absorption [4]. The loss, including evaporation, is generally considered as a whole.

In ordinary navigation canals with extremely slow currents and passing through good firm soil the losses are probably little greater than in reservoirs.

On some French navigation canals the losses are stated by Parker [5] to have averaged 90, 48, 40 and 12 cubic feet

Irrigation Works in India and Egypt, Buckley

Proc Inst C E, Vol CCVII p 107

Irrigation Engineering, Chap XV

Sometimes the word "seepage" is used. This really means leakage or percolation but is used to include absorption

Control of Water, p 738

per second per million square feet, the channels being in fissured rock, chalk, gravel and alluvial soil respectively. The maximum loss was 3 to 5 times the average. The losses were in the summit levels and the water was supplied from reservoirs or at least was, in nearly all the cases, extremely free from silt.

In a river or canal with a fairly rapid current the case is very different from that of a reservoir. There may be no silt deposit either on bed or sides; or the deposit in the channel may be sandy.

Some years ago experiments were made in the Punjab as to the effect of lining watercourses with various materials [1]. The following results and conclusions were arrived at as regards ordinary unlined trenches.

(*a*) The rate of absorption varies greatly, and this is due probably to unequal fissuring of the upper layers of the soil.

(*b*) The rate of absorption in the three hottest months averaged .0571 feet per hour, or more than double the rate (.026) in the three coldest months. The difference was ascribed to the greater viscosity of the water when cold.

(*c*) The average losses with canal water were .0315 feet per hour, or 8.75 cubic feet per second per million square feet. The losses occurred in water only about 1 foot deep. With well water the figures were .1096 and 30.5. The conclusion is that the silt in canal water reduced the losses by more than two-thirds.

(*d*) With canal water the average loss decreased by 40 per cent. (from .0491 to .0293 feet per hour, or from 13.64 to 8.14 cubic feet per second per million square feet) in about four years. This was no doubt due to the effect of the silt. With well water the loss at the end of four years (.2293) was nearly four times as great as at first (.05191). This may have been due to removal of the finer particles of soil by the water,

[1] *Punjab Irrigation Paper*, No. 11 C Lining of Watercourses to reduce absorption losses. Experiments of 1908—1911.

but the experiments were made at only one place, and were not conclusive.

The losses in Punjab canals and their branches and distributaries of all sizes, have been found to be from 2 to 60 (generally from 5 to 20) cubic feet per second per million square feet of wetted surface. Details are given in the accompanying statement. Some figures obtained in Egypt for main canals agree fairly with the above. On the Sone canals in Bengal the loss has been found to be 10 cubic feet, and on Californian canals 9.4 cubic feet, per second per million square feet.

Channel	Nature of soil	Mean depth of water in channel (feet)	Loss per million square feet of wetted surface (c ft per sec)	Remarks
Main Lines				
Upper Bari Doab Canal	Shingle and Sandy Soil	6	9.7	Fairly reliable estimates based on discharge observations
Sirhind Canal	Sandy Soil	7	9.0	
Branches				
Upper Bari Doab Canal	Loam		2.2	
Sirhind Canal	Sandy Soil		5.2	
Distributaries				
Upper Bari Doab Canal	Loam		2.3 to 4.4 (average 3.3	Somewhat rough estimates
Sirhind Canal	Sandy Soil		5 to 12 (average 8.0)	
Watercourses				
Upper Bari Doab Canal	Loam		3.3 to 20 (average 9.4)	
Sirhind Canal	Sandy Soil		7 to 60 (average 22)	

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Watercourses				
Upper Bari Doab Canal	Loam		3.3 to 20 (average, 9.4)	
Sirhind Canal	Sandy Soil		7 to 60 (average 22)	

The canals and branches received either no deposits of silt or deposits consisting largely of sand. The distributaries received deposits of silt which was only occasionally cleared away. The watercourses receive deposits but they are frequently cleared out by the cultivators. This is perhaps the reason why the rate of loss in the watercourses was nearly three times as great as in the distributaries of the same canal. On the Sirhind canal the distributaries have more branches than on the Bari Doab canal and the watercourses are smaller. This may partly account for the different relative losses in the two cases. The sandy nature of the soil on the Sirhind canal accounts for the general higher value of *c* on that canal.

The depth of water in branches is less than in main lines, less again in distributaries and least in watercourses. The effect of depth on loss is not apparent, especially having regard to the losses of water in the trenches mentioned above.

In Idaho in the United States, the losses were observed in a great number of canals of all sizes. The average loss was found to be 1.21 feet per day and the maximum loss no less than 6.32 feet [1].

On Bombay Canals (Sind, Deccan and Gujrat) 48 pairs of discharge observations were made in 1918—1919 in order to ascertain the losses. The results were as follows, the losses being in cubic feet per second per million square feet of wetted area

Approximate depth of water (feet)	5 to 9	4 to 5	1.5 to 3.5
No. of observations	11	7	27
Mean loss	11.75	24.6	10.1

The depths are not given in the statistics but have been roughly calculated from the sectional areas and wet borders. The high figures in the middle group seem to be accidental, or in some cases due to wind during the velocity observations. There is little to indicate that the depth of water has much effect on the loss.

[1] *Irrigation Engineering*, Davis & Wilson, p 34.

On the main line of the Sirhind Canal in 1914 and 1916 the following losses were observed by Nicholson [1].

Mean depth in reach	6.5	8.3	11.2	11.3	feet.
Loss per million sq. ft.	5.3	8.8	8.7	12.1	c. ft. per second.

Observations on other canals in India, Europe and America show that a rough average loss is 1 foot per day — the range being generally from .4 to 1.7 feet — but with occasional high losses such as 4.7 feet in Bombay in porous soil and 6.4 feet in Colorado [2].

In a paper on the Absorption Losses of Punjab Irrigation Canals [3], the losses which have somewhat recently been occurring in the main canals and branches mentioned in the above statement and in the main canal and branches of the Lower Chenab Canal, are discussed and it is sought to show that in channels discharging more than 500 cubic feet per second the loss is approximately proportional to the depth of water (D). The chief figures adduced are those of losses taken from the official annual statistics concerning the working of the canals The statistics, as has been pointed out [4], are not reliable The difficulties involved in obtaining an accurate series of discharges have been mentioned above, (Chap I., Art. 2), To get the correct differences between two series is still more difficult The distributary discharges — only approximately accurate — are all added up and deducted from the head discharge of the canal or branch. The losses are given for the summer and winter seasons of six months each Omitting the figures for certain years — some of these were absurd and were rejected in the paper under discussion — and considering only the years 1899—1909 the following figures result.

[1] *Proceedings, Punjab Engineering Congress*, 1917
[2] *Irrigation Pocket Book*, Buckley
[3] *Proceedings, Punjab Engineering Congress*, 1916.
[4] *Irrigation Works*, Bellasis

Canal	Winter figures (per cent. of summer figures)		
	Loss of water (S)	Mean discharge (Q)	Mean depth of water (D)
Upper Bari Doab . .	64	50	63
Sirhind	67	79	86
Lower Chenab	75	82	88
Mean	69	70	79

These figures can only be considered as approximations. Those in column 4 have been calculated from those in column 3. It can be said to be probable that, with D reduced to 79 per cent. of its summer value, S was reduced to 69 per cent. of its summer value. But it is by no means clear that S is simply porportional to D. The height (H) of the water-level in the canal above the water table in the soil, is not great and the loss of water may depend largely on H. In many places H was quite small. On the Lower Chenab Canal channels, opened only in 1887, H — at first great — was rapidly decreasing from year to year. The loss must also depend on the viscosity of the water and this is least in summer.

In the case of a channel — whether canal or river — traversing swampy ground there may be a gain of water instead of a loss.

In the case of the branches, Upper Bari Doab Canal — see above table — the mean depth of water seems to have been not more than 3.5 feet which is small for a branch. This is somewhat in favour of the contention put forward as to D but perhaps also in favour of the loss depending on H.

A formula proposed by Davis and Wilson is

$$S = C\sqrt[3]{d}\,\frac{A}{4{,}000{,}000 + 2{,}000\sqrt{V}} \qquad (3)$$

where A is the area of the wetted surface [1].

[1] *Irrigation Engineering*, Chap XIII.

The values of C for clay, clay loam, medium loam, sandy loam, coarse sandy loam, fine sand, medium sand, coarse sand and gravel, are given as 12, 15, 20, 25, 30, 40, 50, 70 respectively.

Eliminating all reference to V and putting the denominator as 4,000,000 and taking C as 20 the losses per million square feet are as follows:

D =	8	2	1	feet
Loss by above formula =	10	7.95	5	c. ft. per second per million square feet.
Loss according to a formula in the paper discussed above =	9.6	4.8	1.2	

When D = 1 these last results totally disagree with the figures for small channels — watercourses and trenches — recorded above.

On the Nira Left Bank Canal in the Deccan, Bombay Presidency, the loss has been found to vary from 1.8 to 13.3 cubic feet per second (in 15 out of 20 observations it was only from 1.8 to 4.4 cubic feet per second) per million square feet, the depth of water ranging from 2.7 feet to 6.7 feet. The loss generally varied nearly as the depth of water. It is however explained that the geological formation consists of lava and soil in horizontal layers and that most of the loss is from the sides of the channel. Other observations were made in the Deccan but generally the channels were somewhat new or the discharge very small or the water-level on the day of observation had only been temporarily raised. The figures corresponding to the above ranged from 1 to 25 [1].

It was found in the Deccan, that only a thin film of silt is formed on the wetted surface. The film is "very perishable and only persists when covered continuously with water". It is suggested by Inglis that a very high velocity will tend to reduce the film. The Deccan silt is in "an excessive state of division". No deposit occurs where the velocity is more than .5 V_0.

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[1] *Bombay Engineering Congress*, 1922. Paper No. LXVIII (Inglis).

Sometimes losses are reported merely as percentages of the discharge. The wetted areas should always be given.

In the Punjab, in designing canals it has been usual of recent years, to assume the loss as 8 cubic feet per second per million square feet in all main and branch canals.

The Agra Canal, United Provinces, India, has a slope of only 1 in 10,000 and consequently a low velocity. It has gradually received a deposit of fine silt and has become more water-tight than ordinary Indian canals.

It will be clear from the various figures given above that the loss varies greatly and must be studied in each case. Exceptionally high losses are to be expected when the subsoil is specially porous

In designing a canal the dimensions should first be arrived at approximately, the loss of water calculated and then the channel finally designed.

Art. 3. Hydro-Electric Works. Works for utilising water power are — except for some old installations — all hydro-electric. The water does work in proportion to its volume and the height through which it falls. A horse-power is 33,000 foot-pounds per minute or 550 foot-pounds per second. The weight of a cubic foot of water is 62.5 lbs. If a discharge of 22 cubic feet per second falls through a height of 500 feet, the work which it can do is $\frac{22 \times 62.5 \times 500}{550}$ or 1250 H.P. There may be a small volume of water with a great fall or a great volume with a small fall, these terms of course being relative

The water is sent down a pipe or set of parallel pipes, also called a "penstock" The pipes start from a small reservoir called a "forebay" and terminate in the "power house" which contains the turbines.

The steeper the pipe-line the less is its length — for a given fall — and the less the loss of energy in it. Bends and irregularities of course cause loss of head, as in any other pipe, and should be strictly limited to what is unavoidable. The pipes are made of large diameter — reduced where the water is delivered to the wheels — so as to reduce the

loss of head. The pressure in the pipes increases with the depth below the forebay. The greater the pressure the greater must be the strength of the pipes.

After passing through the turbines the water passes back to the stream through the "tail race".

The difference in level between the off-take of the canal which conveys water from the river to the forebay and the junction of the tail race with the river, is the gross head. The canal and the tail race should have flat gradients so that the working head — from the forebay to the power house — may differ as little as possible from the gross head. The greater the cross-section of the water-way and the smoother the channel, the less the gradient will be, for a given discharge.

A "high" head is one which is more than 500 feet, a "low" head from 2 or 3 to 80 feet, other heads being classed as "medium". High heads can only be obtained in country which is hilly or mountainous. With high heads the Pelton wheel (or impulse turbine) is used, with low heads the re-action turbine, with medium heads various kinds of impulse wheel. A very high head can be divided andan intermediate power house arranged for.

With the re-action turbine the water after leaving the wheel casing, enters a "draft tube". The water is still doing work. It is discharged below the level of the water in the tail race. The working head is measured down to the level of the tail water. With the Pelton wheel it is measured down to where the water is discharged into the wheel.

The fall may be obtained from a single natural waterfall, or from a series of falls or rapids It may be increased by means of a weir.

In mountainous country one or more reservoirs may be constructed at great elevations and rainfall collected in them or natural lakes utilised. The water, is drawn off by canals from the reservoirs instead of from a river.

In any case the canal from the off-take can only extend as far as there is ground high enough — and otherwise suitable — for it. Hence the pipe-line may have to be long.

A low or medium head can be obtained by throwing a weir across a river or by making a cut across a loop (Fig. 62). When a weir is constructed it may be necessary — if no lock is made — to provide a flume which can be used as a "log-run".

In a case where a river divides into branches, a side channel can be used as a canal, a weir and power house being built near where it again joins the main river.

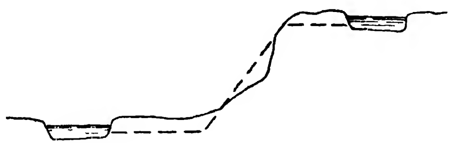


FIG 61.

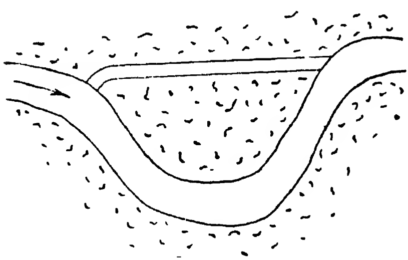


FIG 62

Sometimes — in order to obtain a good fall — the power house is placed, not in the same valley as that in which the river flows but in a neighbouring valley, the water being conveyed across the watershed.

When the fall is small and the volume of water large, heavy works are necessary. The cost of the installation per horse power increases rapidly as the working head decreases.

In the case shown in Fig. 61 the best positions for the beds of the canal, pipe-line and tail-race are obviously somewhat as shewn by the dotted lines. If the steep slope occurred elsewhere the canal would be lengthened or shortened. In some similar cases the pipes have been taken

down a vertical shaft close to the off-take — the power house being thus underground and the tail-race partly in a tunnel, an expensive and undesirable arrangement.

When the pipe-line can suitably begin close to the off-take from the river or reservoir there is no canal, the water passing directly into the fore-bay. When the canal is short it may be of wood, concrete or steel. Sometimes instead of an open channel it is a pipe — wood-stave or other — with low pressure. In the case of a low fall there is a very short pipe-line or none at all, the re-action wheel being simply placed in a wheel pit. The upper part of a pipe-line is not infrequently a "pressure tunnel". In other cases the pipe-line passes through a tunnel.

In cases where the head is low it often becomes less in floods. The upstream water-level in the river may be, at most seasons, held up by a weir or dam, but in floods sluice gates have to be opened so that the upstream water-level does not rise so much as the downstream water-level. The water-level in the tail race, of course, depends on the latter. In some cases a waste weir has been so designed as to produce a standing wave (*Hydraulics*, Chap. VII. Art. 11). The tail race discharges into the trough which is formed upstream of the wave. Thus the working head is increased in floods instead of being reduced [1]. If the river debouches into another, the floods in the latter may cause heading up. In any case the matter requires careful attention at the time the works are designed.

In hydro-electric works the "load" on the turbines — and consequently the quantity of water required — is not constant. It may for instance be far greater during the day than at night. This can be arranged for without much waste — if any — by providing reservoirs of moderate capacity between the canal off-take and the forebay. The forebay — if there is no reservoir near to it — should be of sufficient capacity to deal with sudden changes of load of short period. An escape for surplus water can also be pro-

[1] The possibility of the trough being utilised in this manner was suggested in the first edition of *Hydraulics*

vided there. Such water — where no other escape is practicable — is sometimes taken by a pipe into the tail race.

If the stream supplying the works is intermittent, or changes much in volume with the season, it may be necessary, in order to ensure a steady supply from week to week and from month to month, to provide capacious reservoirs. This is however, impracticable in cases of low head. The storage required would be too great. If a steady supply is necessary it must at times be much less than is available.

In many cases storage is obtained by throwing across the stream not merely a weir but a high dam which forms a reservoir in the bed of the stream. At the same time it is a means of creating a fall. This plan is not suitable in a stream in which a great quantity of material is rolled or carried. The height of such a dam may be limited by the necessity for avoiding flooding of the land upstream of it.

When a dam is constructed the power house can be on either flank of it. Sometimes a hollow dam is built and the power house placed inside it.

In designing a hydro-electric work many alternatives may be feasible. As regards the discharge, it may be desirable to utilise the minimum flow of the stream and no more — or to increase it by means of storage and to utilise only this increased minimum — or to utilise a greater discharge in the seasons when it becomes available.

There is also the question of the amount of fall to be utilised. The site may be such as to be well adapted to a particular fall. An increased fall may easily involve a greater relative cost, because of the greater cost of the dam or canal or pipe-line.

It is obviously impossible to give any rules for the solution of the above and other kindred problems. As in other engineering schemes the initial cost, working expenses and income of alternative schemes must be estimated and compared.

In hydro-electric works it is specially important to rid the water of sand and rubbish. Sand — especially when the water has a high velocity — does great damage to machinery

and also wears away pipes. A sand-trap (p. 136) may be required at the canal off-take. The forebay can be made to act as one and there may be others. Generally no water should be allowed to enter the canal — except at its off-take — lest it should bring in rubbish.

At the off-takes of canals for hydro-electric works, gratings — called in America "trash-racks" — are used in order to exclude floating rubbish, wood or ice. In cold climates hot air is blown against the racks in order to prevent their being choked by the formation of ice upon them.

The Committee lately appointed to make a report in connection with the Water Resources of the British Empire, state in their report that in the absence of facilities for extensive storage it is necessary — where the power is required for manufactures — to consider the minimum power likely to be available towards the end of the longest drought, but that where the power is for mining, agriculture or forestry there are possibilities that flood supplies can be used for seasonal operations. They also state that the investigations concerning water power sites should extend over many years and should include records of discharge of streams, that such investigations will be useful in connection with irrigation, navigation, water supply, floods and land reclamation, and must include contour plans of sites and profiles along the entire power reach of the river and along the bank, and must also include studies of lakes and possibilities of their utilisation and inter-connection. They further state that it is not suitable merely to develop the most obvious sites; that storage conditions should be specially investigated — the maximum storage possible being recorded — and that without complete surveys the capacity of a river cannot be accurately judged.

In most great countries, general state control over water-power schemes is being adopted and enquiries are on foot to obtain detailed information as to the sites available. The largest installation in Great Britain is that at Kinlochleven where the head is 920 feet and the horse power 30,000. The greatest amount of available power is in the United

States and in Canada. There are also great possibilities in India. New Zealand offers a good field. The rivers of Ireland have steep falls in the last few miles of their courses and this gives good sites for water power.

A notable hydro-eletcric scheme is the Tata power-supply

FIG 63.

work for Bombay. The reservoirs (Fig. 63) are in the Bombay hills or Western Ghats, which rise in horizontal layers of basalt and trap rock to heights of 2,000 to 3,000 feet above the sea. The country slopes very steeply to the west. To the east the slope is more gradual. The masonry dams constructed to make the Shirawta and Walwhan Lakes

are on their east sides. The lakes are respectively 2162 and 2084 feet — they are connected by a tunnel — and that of the Lonawla Lake 2051 feet above the sea. The Lonawla is a "monsoon lake". It could not have been used for very large storage without very costly dams at each end. Its capacity is sufficient to supply the balance of the power demands during periods of the monsoon when the daily rainfall does not suffice. From it a 2-mile canal or "duct" leads to the forebay which is formed by a masonry dam across a narrow valley During the non-monsoon period of the year the supply is drawn from the other two lakes, the duct issuing from the Walwhan and joining the Lonawla duct.

The pipe-line is 12,520 feet long and descends 1,725 feet — to the west through a gap in the hills — the gradients being in many places precipitous. In the first part — 8,206 feet — of the line the fall is 570 feet and there are two lines of steel pipes of diameter decreasing from $82\frac{1}{2}$ inches to 72 inches, the upper section being riveted and the lower section lap-welded and the thickness ranging from $\frac{3}{8}$ to $1\frac{3}{8}$ inch. At the end of the first part of the line there is a distributing pipe. In the second part — 4,314 feet — of the line the fall is 1155 feet and there are 8 lines of steel pipes of diameter varying from 42 to 38 inches, lap-welded, the thickness ranging from $\frac{1}{2}$ inch to $1\frac{1}{4}$ inch

The rainfall on the catchments has been already stated (Chap. II., Art. 7). The works are capable of developing 100,000 H.P.

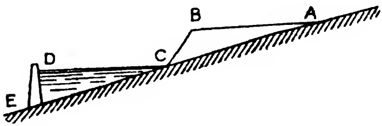
The forebay is of limited capacity and care has to be taken in regulation, the reservoir gates — whether they are being opened or closed — being given a "lead" that is operated a considerable time before the supply to the turbines is increased or reduced at the forebay. Otherwise water at the forebay might either run short or have to be wasted [1].

In the hydro-electric works for the supply of Barcelona

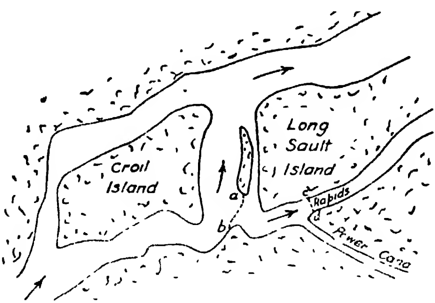
[1] *Proc. Inst. C. E.*, Vol CCVII. p 29.

there is a canal AB (Fig. 64) a pipe-line BC and a reservoir CD, with power houses at C and E. The hatched line is the river bed. The canal is taken along higher ground. Lower down the river the above arrangement is twice repeated [1].

In hydro-electric works near Amritsar in the Punjab the power is obtained from a fall of 5.8 feet in a branch of



the Upper Bari Doab Canal. The work is designed to utilise half of the discharge — which is 2100 cubic feet per second — and the power is used for pumping water from the soil

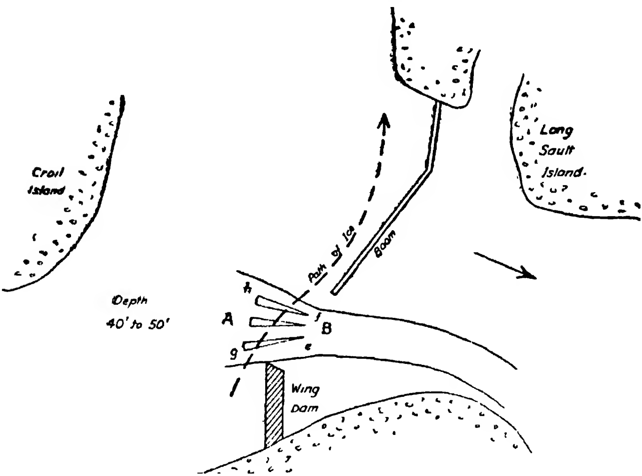


in order to alleviate its water-logged condition. The water is then used for irrigation [2].

Near Asmara in Eritrèa a flow of about 2 cubic feet per second — obtained from small hill reservoirs — is pumped up about 130 feet to the crest of a ridge whence it falls down the opposite slope, developing some 300 H.P — about 10 times the H.P. used in raising it – and is subsequently used for irrigation [3].

[1] *Proc Inst C. E.*, Vol. CCXIII p. 295.
[2] *Proc. Inst C. E*, Vol. CCXII p 66.
[3] *Times Engineering Supplement*, 28 April 1923.

In the case of the Massena hydro-electric works on the river St. Lawrence, ice entering the power canal (Fig. 65) reduced the water supply. Ice-jams lower down the river headed up the water and so raised the water-level in the tail-race, reducing the head by as much as 10 feet. The power house with a capacity of 90,000 H.P. had frequently to be shut down. The problem was to prevent ice from going down the right-hand channel. It was solved by means of a deep channel with wedge-shaped ridges in its upper part



(Fig. 66). The ridges were submerged to a depth of 15 feet at A and 9 feet at B Between the ridges the water was 40 feet deep. The lower water flowed down the deep channels while the surface water — carrying with it floating ice, weeds and timber — followed the course shown by the dotted line. The cross flow had a velocity of 8 to 10 feet per second along *gh* but only 1 to 2 feet per second along *ef*. A boom *ab* — consisting of a line of heavy scows drawing 4 or 5 feet of water — was also constructed in order to prevent the diverted ice from going down the right-hand

channel. A submerged weir *cd* was made by dropping large stones from barges, by means of a derrick, on to the hard bottom of the stream in order to reduce the surface slope over the shoal so that the boom would be safe [1].

Combinations of hydro-electric works with other works in which water is used, are adopted to some extent. Irrigation canals afford numerous sites where there are falls of several feet but these are of limited use because of the want of steadiness in the supply. Branch canals are at certain seasons run in turns, and whole canal systems are closed for repairs or because of lack of demand for water after heavy rain. Nevertheless there are several cases in India where power derived from falls in large canals is used for factories, lighting and electric fans. Power water can sometimes be used for irrigation without difficulty.

In navigable canals or canalized rivers, power stations are sometimes erected at the weirs. In Germany some such power plants are suited to discharges which are only available on 80 to 100 days in the year [2]. Any increase in the number of weirs in such a channel may be desirable for navigation but is inimical to power.

There is an instance of an aqueduct, carrying a supply of water for a town, being tapped for power purposes the water again entering the aqueduct lower down.

Art. 4. Irrigation Canals. The headworks of a large irrigation canal generally consist of a weir — which may be provided (Chap. IV., Art. 1) with sluices — across the river, and a head "regulator", provided with gates, for the canal. There are however some canals, which have no works in the river (Art. 1). If a canal is fed from a reservoir the headworks consist simply of a sluice or sluices. Sometimes the reservoir is formed by throwing a dam across a considerable stream. In this case the canal is not necessarily taken off from the reservoir. The water from the reservoir may be sent on down the stream and a canal — with a weir and head regulator — be taken off lower down. The great Assuan

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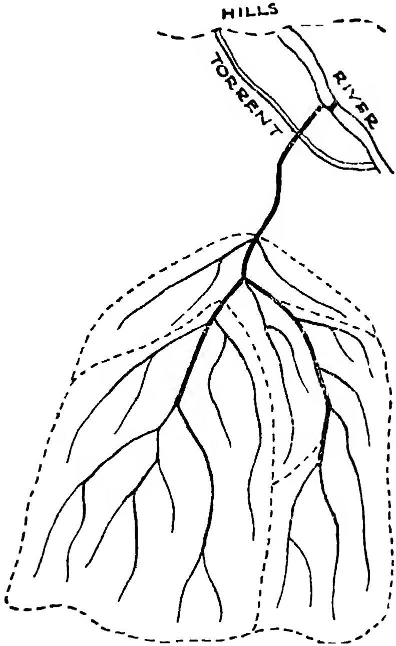
[1] *Canadian Engineer*, Vol. 39, p 545.

[2] *Elektrotechnische Zeitschrift*, Vol. 41, p. 765.

reservoir in Egypt is of the above type and is formed by a dam across the Nile.

A canal must bring the water to within reasonable distance of every part of the area to be irrigated. Unless the area is small or narrow the canal must have branches and



distributaries. In America these are known as laterals and sub-laterals. A general sketch of a large canal is given in Fig. 67.

From each distributary, watercourses — in America farm ditches — take off at intervals and convey the water to the fields. The dotted lines in Fig. 67 show the "irrigation boundaries" of the distributaries or groups of distributaries, that is the lines up to which they are to irrigate. Such

lines are in low ground and generally follow drainages.

It is not often the case that the whole tract covered by a system of canal channels is irrigated. In the case of a canal fed from a river, the land near the river is often high or broken and the main canal runs for some distance before it reaches the tract to be irrigated. Again, within this tract there are usually areas too high to be irrigated.

The channels of an irrigation system should run on high ground. In the case of a distributary, this is necessary in order that the watercourses may run downhill, and since the water in the canal and branches has to flow into the distributaries, the canal and branches must also be on high ground. Another reason for this is that the channels may keep away from the natural drainage lines of the country and not obstruct them. Also a channel in high ground is cheapest and safest.

Generally a tract of country possesses more or less defined ridges and valleys. When the ridges are well defined, the distributaries follow them closely. They may deviate slightly on one side or the other from the top of the ridge in order to secure a more direct course, or if any part of the ridge is so high as to necessitate deep digging. In the case of a main canal or a branch the extra cost of deep digging or of a circuitous course, may be great and considerable deviation from the ridge may be necessary. In any case if deviation from the crest of the ridge is such as to cause the banks of the channel to interfere to any great extent with the run-off of rain water and to necessitate the construction of a syphon, due weight must be given to this.

The alignment of a channel should be central with reference to the area dependent on it. Suppose that a distributary irrigates a belt of country 4 miles wide. If it runs down the centre of the belt the distance from the distributary to the centre of the strip on either side is 1 mile; but if it runs down one side of the belt the distance is 2 miles and the watercourses must on the whole be twice as long. Simi-

larly the distributaries are shortest when the canal is central.

If, however, the tract to be irrigated by a canal is all part of one river valley it may have a general slope towards the river. In this case the canal runs along the highest side of the tract, that is the side away from the river, and its distributaries all take off on the side next the river.

The angles at which the channels branch off have to be considered. If branches were taken off very high up the canal and ran parallel to and not far from it, there would be an excessive length of channel. But neither should the branches be so arranged as to form a series of right angles. In the case shown in Fig. 68 the size of the main or central canal would be reduced at the point A. By altering the branches

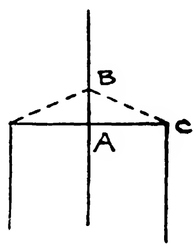


Fig 68

to the positions shown in dotted lines their length is not appreciably increased while the length AB is made of the reduced instead of the full size Moreover the course BC is more direct than BAC and this may be of the greatest importance as regards gaining the necessary "command". When a channel bifurcates, the total wet border always increases and there is then a greater loss from absorption. The water is always kept in bulk as long as possible. If the alignment of a branch is somewhat crooked it does not follow that straightening it — supposing the features of the country admit of this — will be desirable. It may increase the length of distributaries taken off near the bends.

A rule in India is that a distributary ought, when matters can be so arranged, to irrigate the country for two miles on either side of it and watercourses should be two

or three miles long. A distributary should not therefore extend right up to the boundary of its commanded area but stop two or three miles from it.

The rule mentioned in the preceding paragraph is intended to be followed so far as the physical features of the country permit, or to assist in deciding between alternative schemes. It may for instance be a question whether to construct one distributary or two, between two nearly parallel branches. The two-mile rule may enable the matter to be decided or it may influence the decision as to the exact alignments of the branches. The flatter the country and the less marked the ridges the more the alignment can be based on the above rule Sometimes, as in the low land adjoining a river, the ridges are ill defined or non-existent and the alignment can be based entirely on the rule.

In flat valleys, where the ground nearest the river slopes away from it, a canal can — at least at certain seasons — irrigate the low land even if taken off at right angles to the river. But to irrigate the high land near the river and that towards the hills or watershed, a canal taking off higher up the river is necessary. Of course much depends on whether the canal is to irrigate when the river is low or only when it is high, and whether or not there is to be a weir in the river. In Upper Egypt, it is common for a high level canal taking off far upstream, to divide into two branches, one for the land near the river and one for the land towards the watershed, and for both branches to be crossed — by means of syphons — by a low-level canal which irrigates the low ground Similar arrangements sometimes occur on Indian inundation canals

Regulators are usually provided at the off-takes of branches In the case of a distributary taking off from a channel many times its own size there is generally only the head regulator of the distributary but in other cases there is a regulator in each channel at the bifurcation. Thus, when the number of bifurcating channels is two it is called a double regulator.

When a river floods the country along its banks as in parts

of Egypt and of the Punjab and California, it is generally necessary to construct marginal embankments before irrigation can be introduced. The canal may take off at a point where flooding does not occur or it may pass through the embankment, a masonry regulator being constructed to prevent the floods from enlarging the gap and breaking into the country.

A large canal is provided, so far as is practicable, with "escapes" by means of which surplus water may be let out (Chap. IV., Art. 1).

The drainage of the whole tract irrigated by a canal must be carefully seen to. The subsoil water level is raised. The rise near to a canal or distributary is due to percolation from the channel and is inevitable unless the channels are lined. In cases where the water table has risen to within a few feet of the ground further rise can be prevented by drains very near together and perhaps 10 feet deep. The rise at places further away, is often largely due to over-watering or to neglect of drainage Immense damage has been done by this "water-logging" of the soil when irrigation water has been supplied to a tract of flat country. The clearance and improvement of the natural drainages should be attended to but here again more and deeper drains may become necessary.

When a canal is designed the average discharge is calculated from the area to be annually irrigated and the "duty" of the water, that is the area which is irrigated in a year — in circumstances somewhat similar to that of the canal under consideration — by 1 cubic foot of water per second. The average discharge is calculated for those days on which the canal was working during the year. If it was dry for half the year the duty is correspondingly low It varies from about 25 acres to 300 acres

Generally the full supply of a canal can only be obtained at certain seasons of the year. It may be that when the canal is full all the other channels are full. When the canal is not full the branches can be run full in turns. This is in accordance with the principle already stated of keeping the water in bulk.

One principle which should be followed is to make the discharging capacity of each channel — and particularly distributaries — suitable to the area which it has to irrigate. Any excess of supply tends not only to waste of water but to over-irrigation and water-logging of the soil.

It has been suggested [1] that it might be suitable to construct irrigation watercourses to a cross-section larger than would be needed and to let them reduce their size by silting, the silt being more water-tight than the original channel. Cross sections of the form and size ultimately desired would have been constructed at close intervals so as to compel silting to take place. This plan seems to have been adopted in America on laterals (distributaries) — chiefly those of small size — partly with the above object but partly because it would otherwise be necessary to close the laterals for silt clearances during the irrigating season and this would interfere with irrigation. The enlarged lateral is thus a long silt trap but without any local contractions to compel silting [2].

It is explained that it is less costly to remove large quantities of silt now and then, than to frequently remove small quantities. This no doubt applies to the cost of the surveying and supervising establishment as well as to labour In India the conditions are very different and the channels are designed so as to discharge no more than is required. In the case of the American channels it appears that if a lateral silts up to the correct size — the size to which it would have been constructed if the water had contained no silt — it is not then left alone but is again enlarged, if it is expected to silt further, though perhaps a lining of silt is left.

The general principles of design and working are the same for small canals as for large. A small canal may be very like the distributary of a large one.

Canals supplied from reservoirs are generally small. Sometimes a canal supplied from a river passes through a reservoir at an advanced point of its course. During floods a

[1] *Irrigation Works*, Bellasis, Chap V., Art. 2.
[2] *Irrigation Engineering*, Davis & Wilson, Chap XIII. pp. 203 and 222.

high supply is run and the reservoir is filled. This enables the size of the canal to be reduced.

Navigation and irrigation can seldom be combined. In India great expenditure was at one time incurred in making large irrigation canals navigable. The velocity was too great for navigation and new railways took the traffic [1].

Art. 5. Navigation Canals. A navigation canal is sometimes all on one level, but generally different reaches are at different levels, the change being made by means of locks. A "lateral" canal — the most common kind — runs along a river valley more or less parallel to the river. It is frequently cheaper to construct such a canal than to canalise the river. A "summit" canal crosses over a ridge and connects two valleys. A navigation canal requires a supply of water to make good the losses which occur by lockage, leakage, or absorption and evaporation. A canal may be of any size according to the size of the boats which are to be used. Owing to the necessity for keeping, as far as possible, to the "balancing depth" (Art. 1) the water-level is generally above the ground level.

A lateral canal obtains water from the river. The small tributary streams which it crosses may sometimes supply water to it but generally they are at too low a level. Moreover they are apt, during floods, to carry silt Generally they are passed under the canal by syphons Reservoirs may also be required, to hold water for use in dry seasons or in order to fill the canal quickly when laid dry for repairs.

In the case of a summit canal there is often much more difficulty. If the canal can be taken across the ridge at a low level as at A (Fig. 70) it may be possible to arrange for four feeders, as shewn on the plan by double lines, each approximately following a contour line and being supplied by the small natural streams from the ridge. In other cases, as where the canal crosses at B, it is necessary to construct storage reservoirs on high ground.

Losses of water from percolation, absorption and evapo-

[1] For further information concerning irrigation canals see book referred to on p. 136.

ration have been considered in Art. 2. Lockage is dealt with below. The leakage through lock gates is often considerable in old locks in canalised rivers where it is not of much consequence. In other cases it can be kept under control.

In tropical countries weeds grow profusely in canals which have still or nearly still water. Traffic tends to keep them down, but they have to be cleared periodically.

The side slopes of the banks of a navigation canal depend on the nature of the soil. They are generally $1\frac{1}{2}$ to 1, but the inner slope may be 2 to 1 (Fig. 69). The banks are generally $1\frac{1}{2}$ or 2 feet above the water-level, the width of the bank on the towing-path side ranging from 8 to 16 feet, but being generally 12 feet — so that two trains of horses can pass one another — and the width of the other bank 4 to 6 feet. The tow-path should have a slight slope away from the canal

The width of a canal is made sufficient for two boats to pass, and the depth is $1\frac{1}{2}$ to 2 feet greater than the draught of the boats used

The resistance of a boat to traction in a canal is given by the formula

$$R = r \frac{8{\cdot}46}{2 + \frac{A}{a}}$$

where r is the resistance in a large body of water and A and a are the areas of the cross-sections of the canal and of the immersed part of the boat. When A is six times a, R is only 6 per cent more than r. In practice A is never less than six times a This is the real reason why there is room for boats to pass one another. The sides near the water surface often wear away, so that the side slope becomes steeper in the upper part and flatter in the lower part (Fig. 71). The wearing away is due partly to waves caused by wind or by the larger wave caused by the rush of water towards the stern of the vessel [1]. If the wearing away is excessive the banks can be protected, but generally they are merely turfed. For methods of protection including the berm used on ship canals see Chap. IV., Art. 3.

[1] See *The Suction caused by Ships*, Bellasis

Where the soil is specially permeable or where breaches have to be guarded against, puddle is frequently used. Sometimes it is laid, to a thickness of about a foot, over both slopes — and even over the bed — and is covered with a foot of earth to protect it. Generally there are simply puddle walls 2 or 3 feet thick (Fig. 69) in the banks.

Near towns or wherever great expense would otherwise

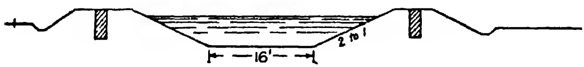


FIG. 69.

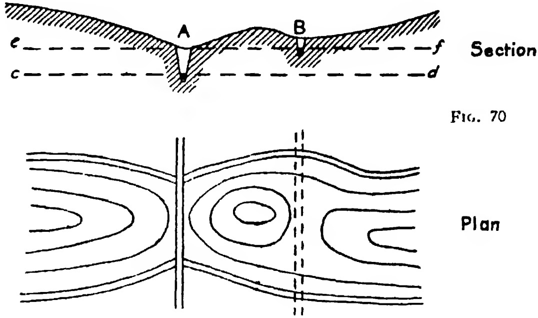


FIG. 70

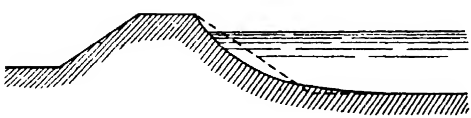


FIG. 71.

be incurred the section of the canal is reduced and its sides are retaining walls with a slight batter

A navigation canal should be provided at suitable places with low-level pipes or "off-lets" by which it can be laid dry for repairs and with a waste weir for getting rid of excess water in times of heavy rain. These discharge into natural streams or drainages. Where a canal is in cutting the outside drains are passed under the tow-path into the canal.

Stop-gates — made like lock-gates — are provided in case of accidents to the lock-gates or breaches in the banks.

Barges are sometimes towed by tugs but the action of the propellers hollows out the canal bed. Their efficiency is also low. Traction by means of small locomotives running along the bank on rails has also been tried. In Germany a rail was laid along the bed of a canal — the upper surface of the rail being 20 inches above the bed — and was gripped by two wheels on a motor boat which thus did the hauling. The result was satisfactory [1].

Small barges are sometimes preferred to large ones because of the shorter time taken in loading and unloading them.

Ship Canals — A ship canal is a barge canal on a large scale. The speed of ships has to be strictly limited to avoid damage to the banks.

The Panama Canal might have been constructed at one level but the cost of this, and the time occupied, would have been double that of making it a summit canal. The water of the river Chagres is impounded to form a lake of great extent that not only supplies water for lockage but itself forms part of the high-level reach of the canal, and ships are able to traverse it at greater speed than in the rest of the canal

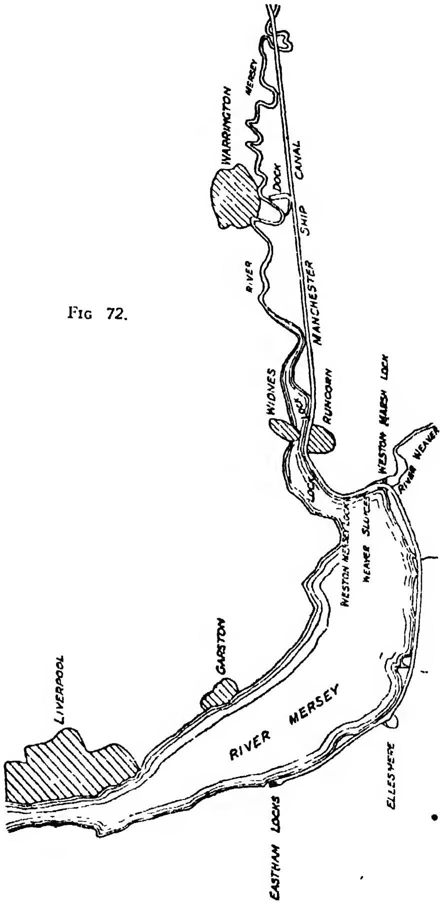
The Manchester Ship Canal (Fig. 72) takes in the waters of the Irwell and the Mersey, and conveys them for several miles. It is thus a canalised river for part of its course. Below that it is a tidal stream, the tide being admitted at Eastham locks where it joins the estuary of the Mersey, and passing out higher up where it leaves the estuary after skirting it. This circulation of water is beneficial to the estuary Tidal openings, provided in the original scheme, had been found to act injuriously by bringing in silt from the river and — in accordance with the Parliamentary powers obtained — had been closed.

When the upper portion of the canal was excavated it was necessary to keep open a channel for the flood and

[1] *Proc. Inst C E*, Vol. CXCIII p 397.

ordinary water of the rivers. Where they were intersected, dams were left in at the end of each cutting. Where the

FIG 72.



canal crossed the river several times in a short distance long new river channels were cut. After each cutting was completed, the dams were excavated and dredged out, and

the rivers were turned through the connected cuttings. The sides of the canal cuttings were raised where the land was low but, as the upper soil was light loam, when the meadows adjacent to the canal were flooded, the water percolated under the longitudinal dams, causing heavy slips and the cuttings then filled with water. In November 1890, 13 miles of the canal were prematurely filled, and in December, 1891, another flood filled 10 miles of cutting. Great damage was caused to the slopes of the cuttings and to the plant. The cost of pumping out the water was serious [1].

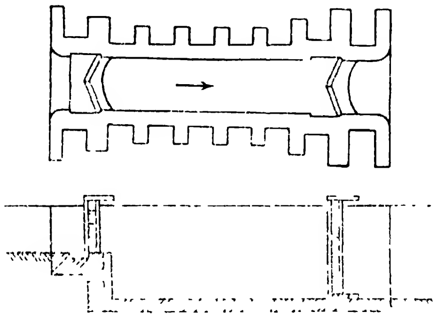


Fig. 73

Locks. — An ordinary lock is shown in Fig 73. The space above the head gates is called the "head bay", and that below the tail gates the "tail bay". The "lift wall" is generally a horizontal arch. The gates when closed press at their lower ends against the "mitre sills"; and the vertical "mitre posts" at the edges of the gates meet and are pressed together. The gate, in opening and closing, revolves about the cylindrical "heel post" — which stands in the "hollow quoin" of the lock wall — and when fully open is contained in the "gate recess".

A lock is always strongly built, of masonry or concrete. The walls have to withstand the earth pressure when the lock is laid dry for repairs. The floor has to withstand the

[1] *Proc Inst C E*, Vol CXXXI p 21

scouring action from the sluices. Regarding the upward pressure of the water when the lock is empty, see Chap. VIII., Art. 2. The floor is usually an invert with a versed sine of about $^1/_{15}$ of the width of the lock. The angle which a gate, when closed, makes with the axis of the lock, is generally about $67\frac{1}{2}°$.

The gates of small locks are generally of wood and are counterbalanced. Those of large locks are of wood or steel, and the weight is generally taken by rollers. Ordinary wood should not be used if the teredo exists in the waters. An iron gate, if enclosed on all sides by plating, is buoyant, and the rollers and anchor straps which hold the upper ends of the heel posts are thus relieved of much weight. The gates of the Panama Canal locks are 110 feet long and 7 feet thick, and the height ranges from 48 feet to 82 feet.

The sluices for filling and emptying a lock are placed in the gates or in the walls. The gates and sluices of large locks are generally worked by hydraulic or electric power.

The heel post of the gate turns at its lower end on a pivot which is bolted to the masonry; at the upper end in a steel collar which is fastened by straps to a counterfort of the side-wall The gate when shut fits close to the hollow quoin (Fig. 74). The centre line of the gate is *ab* and the curves of the quoin and heel post are struck from *a* If the gate turned on *a* there would be great friction against the quoin. It turns on *c* The vertical plane *cd* passes through the centre of gravity of the gate, the planking or sheeting of which is on the upstream side. As the gate opens the heel post swings away from the quoin Its position when open is shown by the dotted lines The side wall upstream of the quoin is set back so that there is water between it and the gate.

All exterior angles in the masonry of a lock are rounded because boats come in contact with them

The length and width of a lock are about a foot more than the length and width of the largest barge to be used on the canal The length is measured from the tail gates to the chord of the arc of the lift wall. There is about 1.5 feet of clearance below the bottom of the barge. The walls

are about 2 feet above high water level. The lift of a lock is generally from 5 to 15 feet. High lifts are becoming more common.

Locks are frequently arranged in flights, the lifts of the locks being all equal. There are, in a few instances, 20 to 30 locks in a flight, the total lift being 150 to 200 feet. By this means the number of gates is reduced, the tail gates of one lock being the head gates of the next, and there is a saving in labour in working the locks. Whether a set of locks can be arranged in a flight with due regard to economy

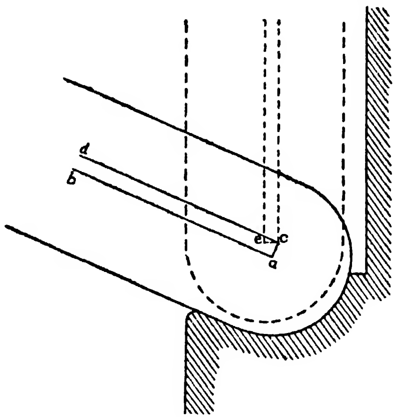


FIG. 74

in construction, depends of course to a great extent on whether an alignment can be found in which the fall in the country occurs chiefly in one locality.

Lockage. — The loss of water caused by the working of the locks, is the quantity which is withdrawn from the uppermost reach of the canal

Let L be the volume of water contained in a lock between the levels of the upper and lower reaches, and let B be the submerged volume of a boat.

When a descending boat enters the lock, it displaces a volume of water, B. This flows back into the upper reach.

The water used in locking the boat down is L — B. When an ascending boat enters the lock the volume B flows down and the lockage is L + B. The average lockage is L. But if an ascending boat is followed by one descending, the lock is only filled once for the two boats and the average lockage is $\frac{L}{2}$. Such an arrangement cannot often be kept up.

When a boat descends through a flight of say, six locks the same water is passed on from lock to lock and the total lockage is L — B. This has been adduced as an advantage — in favour of flights of locks — because at six separate single locks the total lockage is 6 (L — B). But even with single locks, if their lifts are equal, the same water passes on and the total lockage is L — B. If the lifts are not equal one reach gains water and another loses it but this can be adjusted by letting water run through some of the locks. It is best to have the lifts as nearly equal as possible.

Consider a number of boats, and several locks and assume that when a boat has passed out of a lock the gates are left alone until it is seen in which direction the next boat is going.

Let there be 10 boats to go down through 6 locks, whether they are in flights or single The lockage is 10 (L — B). If 10 boats go up the lockage is 60 (L + B). The total lockage for 20 boats is 70 L + 50 B If the 6 locks are single and the boats at each lock go up and down alternately, the total lockage for 10 pairs of boats is 60 L

If L = 10 B, the total lockage for 20 boats (10 up and 10 down) is 60 L in the case where the locks are single and the boats go up and down alternately. In every other case it is 75 L.

Lifts. — In order to save time or to save water, lifts are in many places substituted for locks. The boat is run into a caisson containing water The caisson is then either lifted vertically or moved along an inclined plane. On a canal between Berlin and Stettin a flight of locks with a total lift of 118 feet has been replaced by a lift[1] On the Panama

[1] *Proc. Inst. C E*, Vol. CXCV p 416.

canal a floating caisson, supplied with pumps, is provided in order to close the entrance to any lock when the gate is under repair. It is sunk across the entrance and then pumps the water out of the lock, the water pressure forcing it against the masonry [1].

Canalised Rivers. — These have been described in Chap. V., Art. 3, p. 126. Above a weir in a canalised river the water-level may be higher than the ground level the velocity low and the general conditions very much as in a canal which is wholly artificial except that there are probably some fairly sharp bends. Sometimes, in order to canalise a river it is only necessary to add the weirs and locks and to embank the reaches upstream of them

Part of the canalised river Wey is shown in Fig. 75. AB is part of a cut made to replace the crooked channel CB. FE is a tributary stream Owing to the gain in level effected by the cut and other cuts further upstream, the water in the whole reach AH is above ground level At H there is a lock and at G a weir with sluices and gates, the old channel joining in again below the lock DG is a drain — for the benefit of the land through which it passes — which is taken under the canal by a siphon at M and discharges into the old channel at G KL is a branch taking water for a mill at L, the tail race discharging into the pool LN. The pool does not silt up owing to the disturbance of the water. In floods water is sent round the old channel from G to below the lock H. Similarly as to the old channel CB. Its off-take is above a lock and has a weir and sluices In this way the old reaches of the river are utilised There are mills at many of the locks.

If a weir is made in a straight reach the lock is at one end of the weir If the weir joins the lock at the head bay, the foundations of the lock wall must be designed so as to stand the scour from the weir. If the junction is at the tail bay it may be necessary to extend, in a downstream direction, that side wall of the lock which is next the river,

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[1] *Proc Inst C E.*, Vol CXCIX p 478

in order that boats entering the lock from below may be protected from the swirl and eddying of the water.

Art. 6. Design and Maintenance of Channels. A gene-

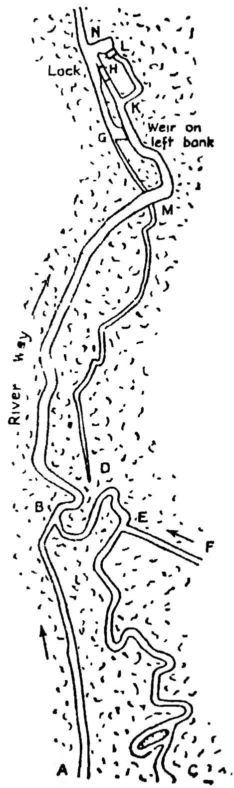

FIG 75.

ral outline of the steps necessary for designing an earthen channel has been given in Art. 1. It remains to consider the matter in more detail. The channels referred to are of earth unless the contrary is stated or implied.

Velocity of Stream. — The general laws governing silting and scour have been stated in Chap. III., Arts. 2 and 3.

Regarding the velocities which different kinds of channels will bear without being scoured an old rule is that streams flowing with mean velocities of 1, 2 and 3 feet per second will sweep along fine gravel, rounded pebbles 1 inch in diameter and slippery angular stones as big as an egg, respectively. This agrees fairly well with figures given by Dubuat when allowance is made for the fact that he considers the velocity close to the bed and not — as is usual — the mean velocity of the stream It implies that the materials are not stuck together by mud or otherwise.

As regards soils, some soft alluvial deposits will stand only .5 to 1 foot per second, soft and sandy soils 1 to 1.5 feet, light soils 1 5 to 2 feet, good ordinary earth 2 to 3 feet and hard earth — as for instance hard clay or soil containing gravel or reinforced by the roots of plants — 3 to 4 feet per second. All the above figures are however, general and approximate. Soils and beds of streams differ greatly. Experience and local knowledge are necessary. Moreover the scouring power of a stream on its bed depends largely on what it is already transporting The above figures are for clear water

Soft rock will stand 5 to 8 feet per second. Hard rock and masonry of good hard stone will stand 20 feet per second even when the water carries along sand and boulders, and instances have occurred in which brickwork has withstood a velocity of 90 feet per second without injury so long as the water did not carry sand

In the large irrigation canals of India and the United States, excluding lined channels or those whose beds are of boulders, the mean velocity V hardly exceeds 4 feet per second In main and branch canals V is usually 3 to 3.5 feet and in distributaries about 2 feet per second.

If the water is charged with silt to the extent and of the kind of that of the Punjab — or even of the Egyptian — rivers appreciable deposits on the bank may occur if V is as low as even 1.7 feet per second. With smaller charges

of silt lower velocities would be necessary to cause deposits, or the deposits would be slight.

Owing to the laws for the bed and banks being different it is necssary to consider them separately.

The banks of a channel — supposed to be steep — are safe as regards scour if V does not exceed the figures, for particular soils, given above or modified in the light of experience and local knowledge.

The above remarks as to scour also apply to the bed of a channel if the water is clear. Let it be carrying a full charge of Punjab river silt, and let D be 5 feet. This corresponds to $V_0 = 2.38$ feet per second. This velocity will just prevent silt deposit. It will just avoid scour if the bed is of fairly coarse sand. If the sand is finer or the material softer or if the charge is less than above assumed, the corresponding reduced value of V_0 can be estimated. If D is 10 feet V_0 is 3.54 feet per second This would in many cases be too much for the banks and they would have to be protected or else the channel re-designed with a reduced value of V. This involves a reduction in D. The width would have to be increased.

Thus in designing or altering a channel, whether or not the velocity is to be high or low relatively to the depth, that is whether or not deposit on the bed is more likely to occur than scour, care can be taken not to make it actually too high or too low, having regard to the banks

If the silt charge exceeds the full charge of Punjab river silt, V must, in order to avoid deposit, exceed V_0 and if this is too much for the banks the procedure can be as just mentioned. In cases — extremely common — where V_0 is less than in the Punjab and other countries mentioned in Chap III. Art 2, an attempt can be made to estimate it.

When the conditions necessary to ensure stability cannot be attained, silting or scour occurs but it does not necessarily occur everywhere. If it commences at the head of a uniform reach the silt charge at once begins to alter. Either a silt wedge or a scour wedge is formed

If silt is being deposited in a canal, it is not at all likely that the trouble can be remedied by running a higher

supply — the quantity of silt entering the canal is then increased — supposing this to be practicable, unless an escape can be brought into play (Chap. IV., Art. 1).

Cross Section. — The side-slopes of a channel depend chiefly on the soil. In good soil the most usual slopes are 1 to 1 for excavation and 1½ to 1 for embankment. In sandy soil they have to be flatter, the side of the excavation is liable to slip down when wet, and both excavation and bank may need slopes of 1½ to 1 or 2 to 1, or even — in pure sand — 3 to 1.

The proper width and height of bank for any channel depend partly on the maximum depth of water in the channel, and partly on the discharge. Given a depth of water of say 8 feet, a breach will obviously be more disastrous with a great volume of water than with a small volume. The following statement gives some figures suitable to the rather light and friable soils of Northern India, but the question is largely one of judgment. Generally a low and rather wide bank is preferable to a higher and narrower one.

Discharge (C. ft. per sec)	Top Width of Bank (Feet)	Height of Bank above F. S. (Feet)
12,000	20	2
8,000	18	2
5,000	16	2
3,000	14	2
2,000	16	1.5
1,500	14	1.5
1,200	12	1.5
1,000	10	1.5
700	9	1.5
500	8	1.5
400	7	1.5
300	6	1
200	5	1
100	4	1
50	3	1

Figs. 76 to 79 show cross-sections recently adopted for very large irrigation canals in the Punjab. When the depth of digging is great (Fig. 76) the inner slope of the canal is broken by a berm carrying a drain. Otherwise much guttering of the earthwork would be liable to occur. When the depth of digging is nil (Fig. 78) there is of course no spoil bank. The earth for the bank is got from pits inside the

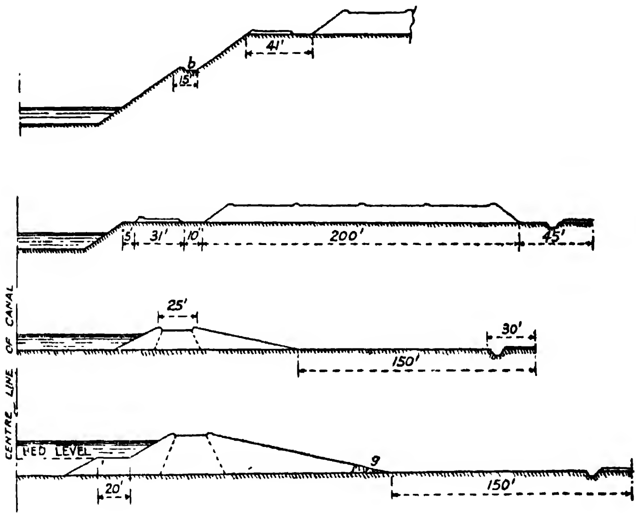

FIG. 76--79

canal (see below) and the bank is set back 15 feet so that there is room for a berm to be formed. When the digging is a minus quantity (Fig. 79) the bank is set back 20 feet.

On the bank a good wide road is provided. In the case shewn in Fig. 77 the road is not allowed to be unduly raised — as it often is on inundation canals and on canals in Egypt and America — the spoil all going behind it. This arrangement facilitates inspection and avoids a long inner slope.

A bank often becomes steep (Chap. III., Art. 3) by silt deposit. The inner slopes of Indian canals thus generally

become about ½ to 1. This is allowed for, the channel being excavated at 1 to 1 but the discharge calculated for ½ to 1. If the silt berm grows excessively or irregularly it is cut down. An excellent bank results.

In large canals the bank, instead of silting, may fall in. If this occurs only locally it is treated by bushing (Chap. IV., Art. 3) but it may extend over a great length, the velocity being too great for the bank though not for the bed.

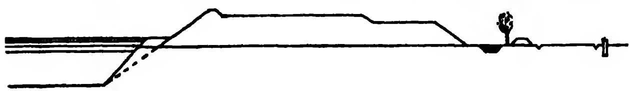

FIG. 80

In some such cases the channel had been designed according to Kennedy's formula but it was not at first realised that the velocity in a large canal has to be limited to what the bank will stand.

Cases like the above — as well as the chance that the canal may be widened — show the great desirability of leaving a berm between excavation and bank. On many irrigation canals — for instance some in America and Egypt and the inundation canals of India — there are no berms. On navigation canals and canalised rivers (Art. 5) there are generally no berms but there is usually a low velocity. In Fig. 77 the berm is very wide and carries the road. The

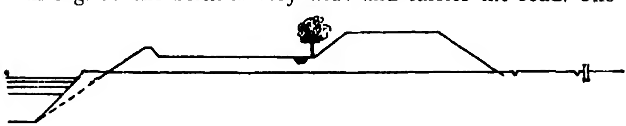

FIG. 81

canal is a big one Generally the berm is between the channel and the road (Figs 80 and 81). These berms become wider if the sides silt up and become ½ to 1. If the soil is sandy, as on some inundation canals, a berm of 20 or 25 feet is often left In Figs 80 and 81 the scale is 6 feet to an inch. The depth of water is 7 feet and the bank, excluding the small raised bank, 2 feet above the water. When the spoil is higher than the road, gaps in it are left at intervals

so that rain water can pass away. The small channels shown are for watering lines of trees. Besides the road on the top of the bank there is a road at the edge of the land acquired.

Regarding the ratio of the bed width W, of a canal to the depth of water D, suppose that in a particular canal, properly designed, the ratio is 6. If the discharge is to be doubled it will probably be best to alter D slightly and W greatly. The ratio obviously tends to increase with the discharge. No rule for fxing the ratio is necessary.

If the upper soil is good and the lower porous, there is a distinct advantage in making the channel wide and shallow so as to confine it to the upper soil. If all the soil is porous the canal can be made narrow and deep so as to reduce the wet border. But generally the thicknesses and levels of the strata vary within short distances.

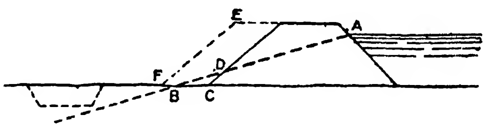

Fig. 82

A channel cross-section of the "best form" is one in which the bed and sides are tangents to a semi-circle whose diameter corresponds with the water level With such a cross-section the sectional area of the stream is a minimum and the velocity a maximum. Such a cross-section may possibly be consistent with the other conditions considered above but this is not often the case. The ratio of depth to width is too great. If the water level does not coincide with the ground level the best form does not give the minimum of exacavation. And in any case the lift of the earthwork is reduced — and therefore the cost — by adopting a greater width and a higher bed level.

In the case of any earthern bank which holds up water there is a line AB (Fig. 82) below which the earth is saturated. If open vertical pipes are inserted in the bank at intervals, the water in them rises to levels which, when connected, form the line AB. AB is the hydraulic gradient.

The better the bank the steeper is the gradient. In a good bank it may be as steep as 1 in 4. The slope DC is wet or moisture is being evaporated from it. The earth below AD is somewhat liable to slip. Or "piping" may occur, small streams of water forming in the bank and gradually taking away soil from inside it, thus causing leaks and possibly a breach. The bank should extend to EF so that the line of gradient may lie wholly within the bank. There is then a large area of dry bank above the wet portion and slips and leakages are improbable.

Hydraulic gradients have received most attention in connection with reservoir dams. In designing ordinary canal banks or even river embankments they have been less frequently considered. The actual gradient in any case can only be estimated. It probably increases in steepness as time goes on and the bank becomes more water-tight. In the case of very high banks on the Upper Jhelum Canal (Fig. 79) the gradient assumed was 1 in 5 and the bank was designed so that there was 3 to 5 feet of dry earth above the gradient line. The outer slope of the bank in Figs. 78 and 79 is 1 in 5.

On side-long ground — even if not steep — there should be a drain running parallel to the line of canal and on the upper side. It prevents water from collecting against the bank or from flowing over the edge of the cutting if there is no bank on the upper side.

Borrow-Pits. — In eastern countries, and others where land is cheap, the earth for banks — in places where the depth of digging is less than the balancing depth — is got from borrow-pits dug alongside the bank. These pits, when dug outside, occupy extra land and often become full of water and breeding places for mosquitoes. As far as possible the pits should be dug in the bed of the channel. They should be arranged as shewn in Fig. 83 the slope of *cd* being that at which the soil will stand when wet. It is not necessarily the same as that of *ab*. One of the two for instance may be sandy. The berm *bc* can be 2 to 10 feet wide. Regarding the arrangements for silting up the pits see Chap. IV., Art. 1.

In wide channels there can be several rows of pits. In sandy or very light soil or in any cases of high banks or of considerable velocity in the channel, the bars left across the pits must be ample and the widths *bc* and *fg* liberal. Otherwise deep channels may be formed in the bed and damage be caused to the side-slopes of the canal. If the water transports little silt, borrow-pits in the channel are less suitable because they may last long and the bars be worn

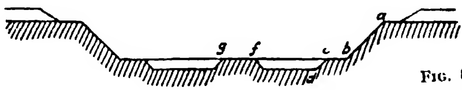

FIG. 83.

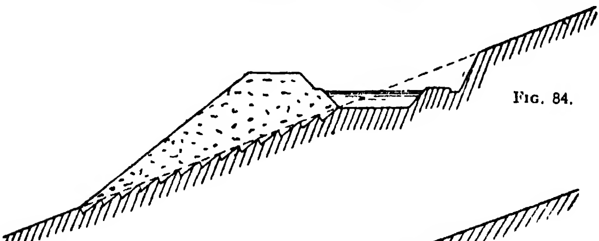

FIG. 84.

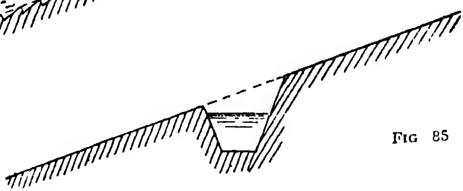

FIG 85

down. Their depth can be restricted unless the soil is very stiff.

Borrow-pits outside the channel should generally be shallow. They should not be dug close to the toe of a slope. The width of the berm is a matter of judgment. It should be greatest in the case of a high and important bank and should be such that the line of hydraulic gradient, and also the line of the slope if produced, passes below the pit (Fig. 82). If the ground level favors flow of water along the pits, bars should be left. It may be feasible to run silt-bearing water through the pits and silt them up.

If convenient the sides of borrow-pits can be stepped

instead of being sloping. All borrow-pits and bars should be set out like any other earthwork.

Special Types of Channel. — If steep side-long ground cannot be avoided altogether the cross-section can be somewhat as shewn in Figs. 84 and 85, a deep and narrow channel being probably the most economical — unless silting is

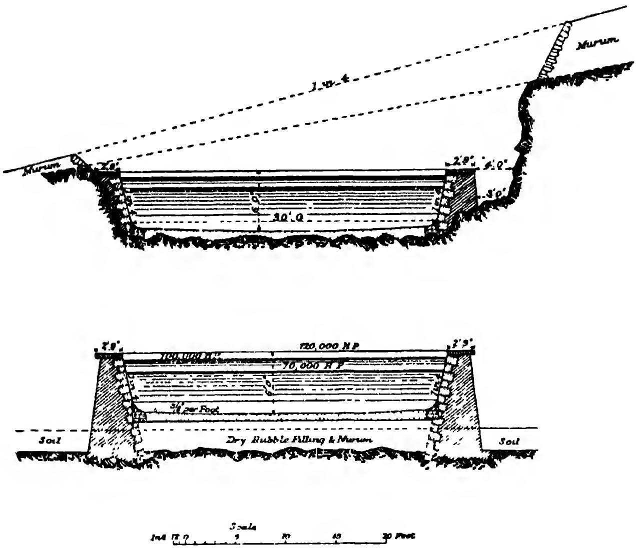

Fig. 86—87

to be feared as a result of such a cross-section — when the water is within soil The side-slopes will not necessarily be so steep as shown The bank shown in Fig 84 — where the water-level is out of soil — is expensive Such a cross-section would not be adopted for any considerable length of channel. In cases of steep side-long ground it is very often best to adopt a channel other than an earthen one. The section shown in Fig. 85 is suitable for soft rock or extremely hard soil. For softer soil the channel can be constructed on a berm as in Fig. 89.

Many canals have to traverse very irregular country especially near their off-takes. This is the case with some canals for hydro-electric works and with some irrigation canals, among recent constructions being several in the United States. In such country canals are frequently made of concrete, wood or metal. The advantages of such chan-

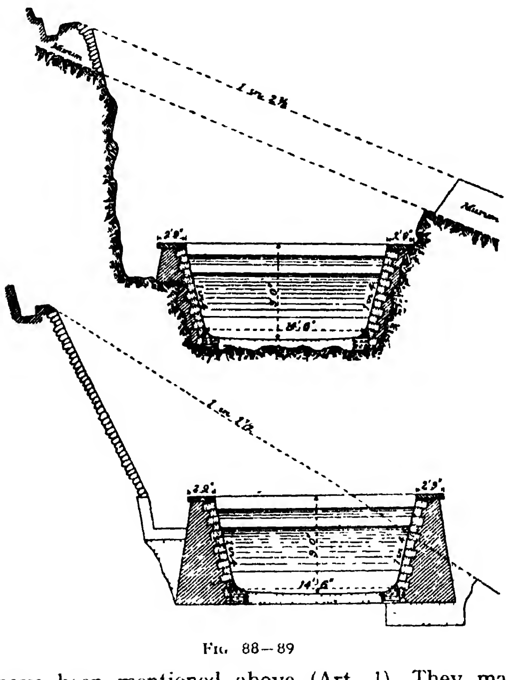

FIG. 88—89

nels have been mentioned above (Art 1). They may be merely used for crossing very low ground or ravines or they may extend for long distances. Not infrequently tunnels have to be constructed.

A well-known case is that of the Tieton Canal in the United States. It traverses steep side-long ground which would be liable to slip if a large cutting were made. The cross-section of the channel is a circle, 8 ft. 3½ ins. in dia-

meter, with the upper part removed, so that the depth is 6 feet. It is made of reinforced concrete 4 inches thick and the sides are tied together by iron bars which run across the channel above the water.

In the Santa Ana Canal the channel consists for 2½ miles of a flume made of wooden staves.

Several of the cross-sections adopted for the canals of the Tata hydro-electric works are shown in Figs. 86 to 89. Here one great object was to prevent loss of water. The "murum" shown is disintegrated trap rock.

One type of steel flume of semi-circular section — diameters 15 inches to 14 feet — is made up in 6-foot lengths

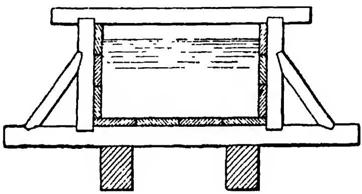

FIG 90

which can be readily joined together, V-shaped channels on each length fitting into grooves in the next. There are also pieces designed for supporting the flume at the joints [1]. A simple form of wooden flume, requiring no metal work except spikes is shown in Fig 90. The figure shows one of the cross-frames which may be a few feet apart The cross beams under the floor are of course nearer together. The flume may be of any convenient size. If the span is great, piers or trestles can be used as intermediate supports. Woodwork should be creosoted or painted unless it is not worth the expense Otherwise it may last only 10 to 15 years.

Cross-sections of tunnels used in the Tata hydro-electric works are shown in Figs 91 and 92 and of typical tunnels — in rock and soft ground respectively — in Figs. 93 and

[1] *Proc. Inst C. E*, Vol. CXCIII, p. 443.

94. Figs. 95 and 96 shows sections used for ducts which have to be roofed over.

A tunnel made in rock may be extremely rough. By lining it with concrete its sectional area is reduced but owing

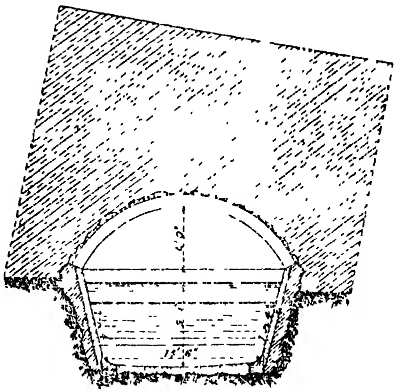

FIG. 91

to the greatly increased smoothness of the channel the discharge is probably increased

Linings — It is highly desirable to reduce the losses of

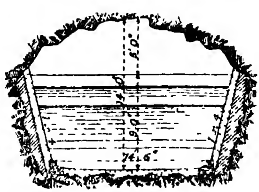

FIG. 92.

water. In the case of irrigation canals the constant escape of water into the ground raises the level of the water-table (pp. 21 and 166) and causes water-logging of the soil, and in all cases wastage of water causes very heavy loss. In power schemes the water may be of very great value. The

remedy is to line the channels. Puddle linings are referred to in Art. 5, silt linings in Art. 4.

In the Punjab, puddle has recently been used on a few of the irrigation channels and is likely to give good results but the channels cannot be kept closed long enough for much progress to be made with linings of any sort. In India

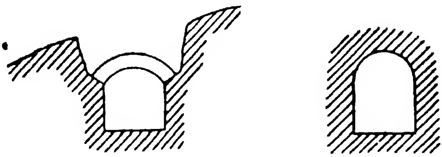

FIG. 93

and America linings of cement concrete $1\frac{1}{2}$ to 6 inches thick and others of cement mortar $\frac{1}{4}$ inch to $1\frac{1}{2}$ inches thick have been used. In some cases great savings of water have been reported.

It is highly important that concrete used for linings should

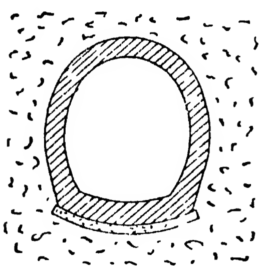

FIG. 94

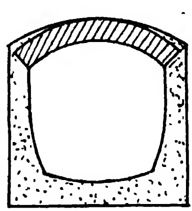

FIG. 95

be free from voids and of the best quality. Special experiments should be made, in order to determine the best ingredients and proportions, before any extensive lining is undertaken. Contraction joints in the lining are essential.

A channel which is to be lined with concrete should be chiefly in excavation. In embankment, while the work is new, the lining will be liable to crack. It is in any case liable

to crack if too thin. Cracks allow of a considerable escapage of water. The backing of the concrete would be very permeable.

In a proposal to line a new Indian canal with concrete it is pointed out that when the canal is dry, the pressure of the water in the soil might crack and displace the lining. The level of the soil water is high because of an existing canal along the same line.

Maintenance. — In earthen banks, great trouble is often caused by rats and other animals which burrow. The holes may cause bad leakages or breaches. Poison is the best remedy. Another, sometimes used in America, is to spread sand on the bank.

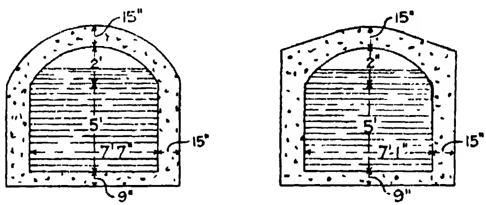

FIG. 96

Vegetation on the inner slopes of the channel should be kept down It shelters rats from birds of prey and also prevents proper inspection of the bank. Thistles and the like can be cut down before they seed. In America they are sometimes encouraged to grow on sandy banks

Vegetation which, by overhanging or otherwise, obstructs the flow of the water, should be rigorously kept down, unless it is desired to encourage the growth of a berm.

An important item in the maintenance of a canal may be the watching of places where breaches are liable to occur. The steps to be taken differ greatly in different countries. In the East, working men have little sense of responsibility and are wholly unreliable as night watchers. Regarding the closing of breaches see Chap. V., Art. 2.

Leakage in a bank can sometimes be stopped by throwing

chaff or other finely divided substances into the water at the site of the leak. In other cases it is necessary to dig up part of the bank, find the channel by which the water is escaping and fill it up by adding earth and ramming. On some navigable canals in France it was at one time the custom to lay the reach dry, when a bad leak occurred, and to dig away the bank and lay slabs of concrete or puddle over the place. This plan was abandoned, and instead of it sheet piles are driven in. They are then withdrawn one at a time, and, if any leakage occurs, the space is filled with concrete.

Snags, and branches of trees and rubbish should be regularly removed from channels. It is best not to let trees grow on the inner slopes.

During heavy rain, holes are liable to occur in banks and roads, especially when new. The holes should be thoroughly opened out by digging and then filled and the earth rammed.

In warm climates weeds may give trouble especially if the velocity of the stream is low. They may completely choke up a channel and in many cases they seriously reduce the velocity. They however, require sunlight and cannot live in water which carries silt. On some canals in Bombay the waves in the reservoirs stir up silt and this is beneficial as regards weeds; as also is a system, sometimes adopted, of letting water down intermittently [1]. Silt deposit favors the growth of weeds especially if the deposit is deep, the weeds having long roots. Weeds can be removed by laying the canal dry. In America they are sometimes dealt with while the canal is flowing by means of a special type of harrow [2].

If the water of a canal is shut off suddenly the water in the soil, draining off rapidly, is apt to wash out material from the banks. These, especially if new, may slip. The banks may also subside to some extent owing to the parts below the water level being soft.

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[1] *Bombay Engineering Congress*, 1922 Paper LXVIII (Inglis).
[2] *Engineering News Record*, Vol. 85, p 319.

Water should be admitted gradually to a canal after closure and particularly to a new canal.

Except in cases where water is abundant the question of leakage through sluice gates or the like should be carefully attended to (Chap. VIII., Art. 6).

The maintenance of hydraulic works requires vigilance. This is apt to be relaxed, in one way or another, in course of time.

CHAPTER VII

FLOODS AND DRAINAGE

Art. 1. General. Floods have already been briefly considered in connection with silting and scour (pp. 51, 59 and 64).

Flood Waves. — The formation and rate of travel of flood waves are considered in *Hydraulics*, Chap. IX. Given the amount of the rise at a point far upstream, the amount of rise at a point further down is greatest when the rise at the upstream point is long maintained so that the flow becomes steady. This case is, however, exceptional. In the great majority of cases the rise of the flood at the upstream point is rapidly followed by a fall. In the paper by Lewis on silt in the Tigris (p. 52, also see Fig. 97) it is shown that in a flood the silt percentage at Amara does not reach its maximum till long after the "peak" of the flood has passed, and a correct conclusion is come to as to the cause of this The rise of a flood is at first due, not so much to movement of "translation" as to wave motion The actual flood water entering the stream is left behind by the wave. Another conclusion drawn in the paper, that the distance between the front of the wave and the arrival of the silt-laden water, must be less in a stream with a steep slope, appears to be correct, though it does not seem that the arrivals can ever be simultaneous. The rises and falls of the water-level are most rapid in torrential streams and become much slower towards the mouth of the stream.

The general shape of a flood wave at two successive sites, is shown in Fig. 98. At the lower site its length is increased to AD and its height diminished. A discharge, observed or calculated at the top of the flood, is of course greater at the

upper site, but the duration of the flood is greater at the

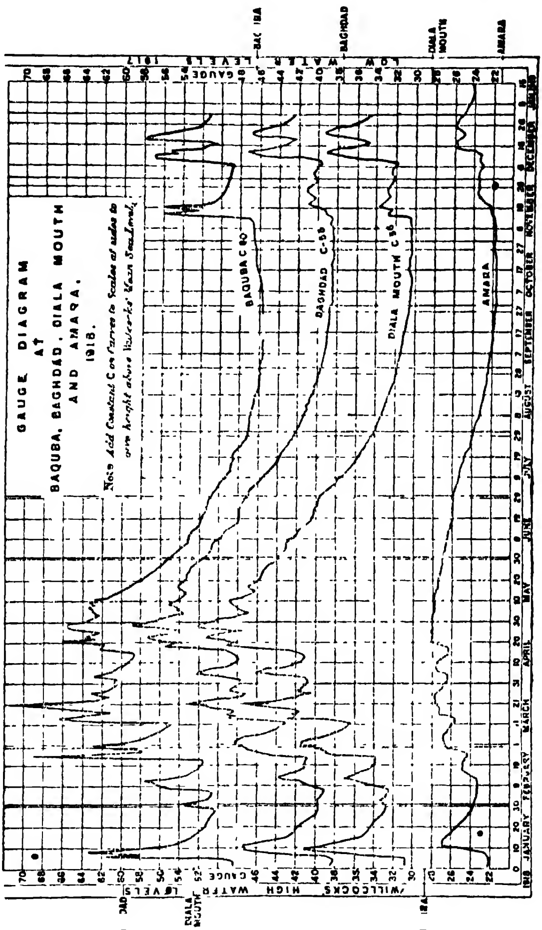

Fig 97.

lower. The total discharge of flood water must, in the absence of gains or losses of water, be the same at both places.

In the case of the bursting of a reservoir dam the slope of the advancing end of the flood wave may be so steep as to give it — from some points of view — the appearance of a wall of water, and its velocity may be extreme. A steep slope and high velocity may also occur in the case of a flood due to rain when the rainfall is excessive and the channel constricted.

Causes of Floods. — The cause of a flood is, of course, in most cases heavy rain but, as will be shown below, the conditions under which it falls may enormously affect the volume of the flood. A disastrous flood occurred at Norwich in 1912, on the occasion of the rainfall mentioned on p. 15. Seven inches of rain in 48 hours in a flat country must cause flooding. The great flood of 1912 in the Ohio and Mississippi rivers was due to heavy rain falling on a layer of ice.

The chance of the occurence of destructive floods is of course increased when the river channel has deteriorated, owing to the accumulation of silt, as has happened in the cases of most of the Eastern English rivers, or to the washing down of silt and sand into the channel where forests have been felled or lands have been settled (p. 29) or mines worked.

It is generally accepted that the extended use of sub-soil field drains in recent years, has increased the severity of floods in England and other countries. They increase the rapidity of the run-off and also to some extent, its amount. An American writer has disputed this, arguing that when a rain storm begins it must find the soil drier — because of the drains — and that the run-off will therefore be decreased. This appears to be true only of a short storm or of the early part of a long storm. Severe floods are caused chiefly by

rainfall of long duration. In such cases the ground becomes soaked after a time. Thereafter run-off occurs both along the surface and through the drains.

In countries having very cold winters, the ice breaks up in spring and it is liable to cause "ice-jams". In April 1922 the breaking up of the ice on the Dwina released great blocks of ice which choked the stream at Dvinsk and caused flooding of the town with much loss of life. The ice barrier was destroyed by gunfire. As to the effect of frazil ice see p. 137.

In mountainous districts, landslips sometimes occur and block the valley of a stream which then forms a lake. The water gradually rises and eventually flows over the dam and sweeps it away, causing a flood which is of great suddenness and height but decreases very quickly in height as it travels down the valley. In a case which occured in the Himalayas in 1888 the inhabitants of the valleys, from the dam to the point where the river debouches from the hills, were compelled by Government to vacate all habitations below the probable level of the flood, and no loss of life occurred In this case the dam was of great size and the formation of the lake occupied many weeks. Proper arrangements could therefore be made. In a similar case which occured in 1921 at Britannia Beach near Vancouver, British Columbia, the height of the dam was about 70 feet, it broke without warning and there was loss of life and property. Great damage occurred in 1921, through the collapse of reservoir dams at Pachuca in Mexico and at Pueblo in Colorado, the reservoir in the latter case being for irrigation.

Prediction of Floods. — At any place high up on the course of a river, the occurrence of a flood can often be predicted when rain storms — perhaps accompanied by lightning — can be seen to be occurring towards the sources of the stream. For any station lower down the stream and for precise information in any case, the readings of gauges higher up the stream can be telegraphed. If the station is at a great distance from the gauge and if there is railway communication, the readings can be sent by post.

In order to be able to predict the time of the arrival of a

flood at the lower station the reading of a gauge there, and also of that at the upper station, should be taken at frequent intervals. In the case of large rivers and distances of hundreds of miles, the interval may be six or even twelve hours, but in other cases it should be less. If the readings are plotted, as in Fig. 99 oblique lines can be drawn to connect the saliences and depressions, and the time taken by each change can thus be readily seen. When the upper part of the stream is formed by two or more important tributaries there should be a gauge in each. It should, of course, in each case, be so far above the junction as to be outside the influence of the other tributary.

As to what constitutes a flood, the hydrograph of a river

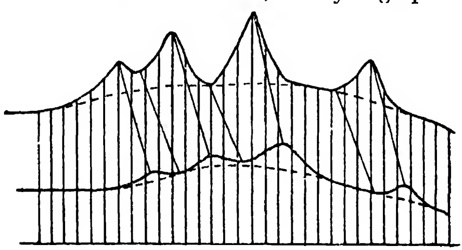

FIG 99

(Fig. 99) is generally such that a line can be sketched as shown dotted. The rises above this line are floods. Leslie's rule for floods in the British Isles is that if all the daily discharges of a stream during the year are ranged in order of magnitude, the discharges of the upper quarter are considered to be floods

In India it is sometimes arranged that a telegram shall, in the low-water stage of the river, be sent from the upper station when a rise of 2 feet occurs in twenty-four hours or any less period, with a further telegram for any such subsequent rise. The telegram states the exact reading on the gauge and whether the water is rising, steady or falling. This indicates the procedure that may be followed where the telegraph has to be used, but where long and frequent telegrams are not desirable.

By taking into consideration the general laws, stated above, as to the rates of travel of flood waves and by noting the actual times obtained from the diagram, it is possible to arrive approximately at the probable time that will be taken by any change. It is also possible to predict the height of the flood. If it is worth while, an empirical formula can be got out. If there are tributaries, each with a gauge, the matter will be more difficult. Probably the floods in the tributaries will arrive at different times but even in such cases empirical formulae have been arrived at, especially in France, and are mentioned in various volumes of the Proceedings of the Institution of Civil Engineers. In any exact system of prediction, the time over which the change extends at the upper gauge must be taken into account, or else there must be several upper gauges and the readings of all of them be taken into account.

In all cases predictions are liable to be more or less upset if rain falls in the tract between the upper and lower gauges. In very dry weather the speed of a flood wave may be somewhat reduced, and the height to which it rises will almost certainly be reduced

These predictions of floods, depending on the existence of floods higher up, have of course no connection with calculating maximum flood discharges (Art. 2).

Damage Caused by Floods.—The most usual damage caused by floods is simply that due to the flooding, whether in rural or urban districts It is most extensive in tropical or sub-tropical countries. But in any case of high velocity, immense damage may be caused by scour or by dynamic effect. The water striking against walls or other objects may injure or wholly destroy them.

Prevention and Mitigation of Floods —The most usual and most effective method of dealing with floods is to construct embankments along the river, on both banks if necessary. The increase thus obtained in the capacity of the stream exceeds anything which can easily be done by enlarging or altering the channel. Another method is to construct reservoirs for storing the water.

The afforestation or reforestation of river basins (p. 29) is also occasionally undertaken, but is not generally practicable.

Sometimes pits are sunk in an area liable to be flooded so that the flood water comes in contact with permeable strata and is absorbed by them.

The chief of the above methods will be dealt with in Arts. 3 and 4.

Where flood water has to be passed through syphons or other openings or over waste weirs or aqueducts, all that can be done — unless some of the water can be diverted — is to make the work of sufficient capacity. The great difficulty in this case — and in all others — is to estimate, with some approach to accuracy, the greatest volume of the flood water.

Art. 2. Flood Discharges. When the flood water of any stream — whether great or small — has to be dealt with, the discharge which the engineer has to ascertain is the maximum discharge There is nothing which, at least in cases where large volumes of water have to be dealt with, requires more careful investigation

In the case of a stream flowing in a fairly well-defined channel the discharge should be calculated from the cross-sections and slope of the stream The first step in such a case is to ascertain, by local enquiry, the highest known flood level. If this can be ascertained the discharge can be calculated, due regard being had to the irregularities of the channel If a discharge can be observed, the value of Kutter's N can be ascertained Otherwise it must be estimated As to these matters see Chap. I. Art. 2.

In remote or uncivilised districts — and particularly if the flow is intermittent — there may be extreme difficulty in obtaining reliable information as to flood levels. In such a case and — at least as a supplementary measure — in the case of a stream which is ill-defined or highly irregular, recourse must be had to calculations based on rainfall.

Heavy falls of rain in short periods are given on pp. 15 to 17. Other falls of equal — or greater — intensity have been recorded in India, America and elsewhere. There has been a fall of 1.5 inch in 10 minutes on the Upper Jhelum Canal

(Pabbi Hills). In other parts of India there are many instances of falls of 3 inches in an hour. Falls of 8 to 10 inches in 3 hours are not altogether infrequent. The heaviest floods are generally due to such falls. They may ocour even in the driest tracts. In Sind where the average annual rainfall is 6 or 7 inches, 6 inches has fallen in a day. It is even recorded that 20 inches of rain fell at Dorbaji in Sind in 6 hours [1]. The possibility of a great and sudden fall of rain seems to depend on the capacity of the clouds to carry it and not on the wetness of the climate. In other cases of great floods the rainfall has extended over 2 or 3 days, or even longer periods, but the total fall is not generally more than 10 or 12 inches and the bulk of it may have taken place on one of the days.

The discharge or total run-off from the catchment is required at some particular point, for instance the site of a work or the point where the stream leaves the catchment. Suppose the catchment to be small but long and narrow. The procedure can be as follows First estimate the time which the water will take in flowing from the most distant part of the catchment to the discharge site. Let this time be T. Then from figures such as those on p. 17, estimate the probable intensity of rainfall R, which will occur during the period T Assume that the catchment is saturated to begin with or quickly becomes saturated, so that the whole of the rainfall will run off. Then, after once the water from the most distant part has begun to reach the discharge site, the discharge will be the whole of the rain falling on the catchment. T is the minimum duration of a storm which will give a run-off equal to the rainfall.

The direction of the prevailing wind should be considered. If the rainstorm travels down the valley the chief effect is a shortening of the time T. After that the intensity of the rainfall over the catchment is the same whether the storm is moving or stationary.

The following table enables the rainfall on a catchment to be readily converted into the corresponding volume of

[1] *Proc. Inst. C. E.*, Vol. XXVII. p. 256 (Brunton).

rain in cubic feet per square mile. Thus for a fall of 1.5 inches the volume is 960 cubic feet.

Rainfall per Hour (Inches)	Cubic feet of Rain per second per Square Mile
6	3,840
5	3,200
4	2,560
3	1,920
2	1,280
1	640
.9	576
.8	512
.7	448
.6	384
.5	320
.4	256
.3	192
.2	128
.1	64
.09	57.6
.08	51.2
.07	44 8
.06	38 4
.05	32
.04	25.6
.03	19.2
.02	12 8
.01	6.4

Regarding the velocities of the streams, the rate at which the rain water flows over the ground into rills or small sub-sidiary streams is usually taken to be ¼ mile per hour in flat land and 1 mile per hour on steep hill sides. The velocity of the current in the rills and larger streams is generally taken to be 2 to 4 miles per hour. The slopes in the upper part of a catchment are generally steeper than in the lower parts

but the streams are smaller. The velocities may not be very different. The catchment area must of course be mapped in order that the correct area and lengths of stream may be obtained, and it must be closely examined in order that reliable estimates of the velocities may be obtained. In important cases they should be calculated. If possible some should be observed.

In the above method of calculation a certain intensity of rainfall was assumed. The following example shows how this may lead to error. In designing syphons to carry torrents across the Upper Jhelum Canal in the Punjab, the discharge from a catchment area of .88 square miles was found by actual observation to be at the rate of 4,165 cubic feet per second per square mile This is equivalent (see above table) to a run-off at the rate of 6 5 inches per hour The catchment area was among low hills — the Pabbi range — not far from the Himalayas, and the declivities of the rills were very steep. The rainfall of this area has already been mentioned (p. 16). The superintending engineer, Mr. R E Purves, has stated [1] that the discharge observations were reliable, and that falls of rain of an inch in ten minutes occurred not infrequently, even though the fall in twenty-four hours might not exceed 2 or 3 inches

The Chief Engineer of the Punjab did not accept the above figures [2]. He remarked that observations taken under great difficulties as to time and place were liable to error, and he considered that an allowance of rainfall at the rate of 4.8 inches per hour — a rate which had been observed elsewhere — and a run-off of 75 of the fall would be sufficient. He accepted a discharge of 2,000 cubic feet per second per square mile for catchment areas of less than 5 square miles, but afterwards increased the figure to 2,400 cubic feet per second per square mile. He did not overlook the fact that in the designs for the drainage syphons a freeboard of 5 feet had been allowed, and this led to an acceptance of

[1] *Report on the Revised Estimate, Upper Jhelum Canal*

[2] *Revised Estimate of the Upper Jhelum, Upper Chenab and Lower Bari Doab Canals.*

an estimated discharge less than would otherwise have been accepted. It is probable that the figure put forward by Mr. Purves was correct. The discharge was observed by an engineer who was careful in such matters. In order to account for the discharge it would at first seem to be necessary to assume not only that a fall at the rate of 6.5 inches per hour had occurred, but that the whole of it had run off. It is not, however, necessary to assume so much. The ground being saturated, the rain falling in a short period might be reaching the discharge site with little loss. A suddenly increased fall at the rate of 6 inches per hour might then occur, and the water travelling more quickly and with hardly any loss, would overtake that already passing the site. The effect would be greatest if the increased fall occurred near to the discharge site There might be merely a shifting of the position of the storm without a change in the intensity or — what is the same thing — the quick succession of one storm by another The rate at which the discharge was arriving at the site of the syphon might be considerably in excess of the rate at which it was falling on the catchment area, and far in excess of — possibly double — the mean rate at which it arrived in a period of an hour.

The discharge of 4,165 cubic feet per second per square mile appears to be the greatest of which there is any record.

When the original project estimate for the Upper Jhelum Canal was framed, the irrigation engineers had had no experience of small and steep catchments, and no one had suspected that the discharge per square mile would be anything like the above. The sums of money provided for works for the passages of torrents had to be increased in ratios varying from 2.5 to 1 to 6 to 1.

In the above case a fall of an inch in 10 minutes was assumed. At a later date a fall of 1.5 inch in 10 minutes was recorded. Thus in small catchments great allowances must be made. It is in connection with small catchments that the construction of works with insufficient waterway has been most common.

If instead of one narrow valley there are two which meet

above the discharge site, it may happen that owing to the lengths of the valleys and the direction of the prevailing wind, the flood of one stream precedes that in the other. The chances of the two occurring together are probably no greater than the chance of the overtaking flood already considered. And similarly in the case of more than two valleys.

The greater the area of a catchment the less the chance of heavy rain occurring all over it within a short period, and the less the chance of the whole area being saturated. The less, other things being equal, is the discharge per square mile. In a large catchment it cannot generally be assumed that the whole of the rainfall will run off. A run-off ratio must be assumed. The ratios commonly adopted are somewhat as follows.

Steep rocky hillsides	.70 to .90
Ordinary hills	.50 to .70
Undulating country	.35 to .50
Flat country	.20 to 35.

The figures can be increased when the surface is specially hard or frozen, and decreased when it is soft, sandy, covered with woods or vegetation, or cultivated.

Corresponding ratios for Bengal are stated by Williams to be as follows:[1]

Mountainous and rocky country	.60 to 90
Wooded hill slopes	.40 to .60
Undulating laterite ground	.25 to .45
Flat land in delta of Bengal	.10 to .20

All such figures are however approximate. Judgment is necessary in using them. If a catchment is dry even a heavy fall of rain may be absorbed. The figures are roughly applicable to heavy and extensive storms. As far as possible actual discharges should be observed and compared with the rainfall; and figures for other catchments of similar character should be obtained and studied.

[1] *The Engineer*, 29th Sept. 1918.

The changes in the intensity of the rainfall cannot affect a large catchment so much as a small one, but they occur of course and therefore there can be an overtaking or a coming together of tributary streams, as before.

In another of the Upper Jhelum catchments, the Jhaba torrent, the catchment area was 56.45 square miles and the length of the main torrent from end to end of the catchment was 13 miles. The velocity of flow was taken to be 6 feet per second. The time required was 4 hours. The corresponding rate of rainfall is 1.7 inches per hour. This gives, supposing all the rainfall to run off, 1,097 cubic feet per second per square mile of catchment area. The largest discharge actually observed was 515 cubic feet second per square mile. The discharge provided for in designing the masonry crossing — where the torrent passes under the canal — was 1,000 cubic feet per second per square mile. Nothing to speak of seems to have been allowed for run-off ratio. Allowance for increased intensity of rainfall was thus secured.

In a case which occurred nearly 40 years ago the calculations made were not so good. An aqueduct was being designed to carry the Ganges canal across the Kali Nadi. The flood discharge, estimated from the supposed flood-level and cross-section of the stream, was 26,352 cubic feet per second. The discharge, estimated by assuming a fall of about 6 inches of rain in twenty-four hours over the catchment area — then believed to be 3,025 square miles — and a run-off of 25 of the fall, was 114,950 cubic feet per second. This figure was rejected on the ground that the rainfall would not be continuous over so large an area as 3,025 square miles An allowance of 7 cubic feet per second per square mile was made and, a fresh survey having shown that the catchment area was only 2,593 square miles, a discharge of 18,000 cubic feet per second was allowed for. The aqueduct was built, about the year 1875, with five arched spans of 35 feet each, the total area of the waterway being about 3,000 square feet. The length of the piers and abutments was 212 feet, the width of the canal carried over

the aqueduct being 192 feet. In 1884 the aqueduct was partly destroyed by a flood whose discharge was about 44,000 cubic feet per second. In July 1885 it was wholly destroyed by a flood whose discharge was estimated at 132,475 cubic feet per second, but was probably more. The discharge must have been more than 51 cubic feet per second per square mile. The aqueduct was rebuilt with a waterway of about 15,000 square feet. Below the aqueduct there was a very substantial brick bridge which had been standing for a hundred years. Its waterway was only 1,146 square feet. It was not much damaged by the flood of 1884, but the water passed round it, breaking through the embanked roadway or pouring over it. Nor was it destroyed by the flood of 1885.

The rain which caused the flood fell on the 17th and 18th July 1885 and amounted to 20.6 inches. If the period is assumed to be 48 hours the rain fell at the rate of .143 inch. per hour. The rainfall per square mile would be 275 cubic feet per second. The catchment area is flat. If the run-off is taken to be .20 of the rainfall the discharge is 55 cubic feet per second per square mile.

The catchment areas above considered were long and narrow. The shape affects the discharge. If the streams all converge to the centre of the base, their average length is a minimum when the catchment is semi-circular, and is somewhat greater when it is a rectangle with its width equal to twice its length, this last dimension being that in the general direction of the flow from the catchment. If the rectangle is turned so that the length is parallel to the flow the run-off is halved. The less the average length the more rapid is the run-off from the whole area. The discharge from an egg-shaped catchment is perhaps 50 per cent greater than that from a long and narrow one of equal area. Different shapes of course involve different arrangements of streams and tributaries. If the times and velocities in the various streams are calculated as above (p. 201) the question of shape needs no further consideration, but otherwise — and in cases of rough

preliminary estimates — weight should be given to shape.

By careful study of any catchment and observation of actual discharges, it will be possible to arrive at a flood discharge which will in all probability be safe. But unless the proposals are extravagant it will not certainly be safe.

General Size of Catchment Area	Reference Number	Country	Locality
Catchments of less than 10 square miles	1	United States	North Braddock, Pa.
	2	India	Near Upper Jhelum Canal
	3	"	"
	4	"	"
	5	"	"
	6	"	"
	7	"	Nagpur, Central India
	8	N. S. Wales	South-East Districts
	9	"	"
	10	Porto Rico	Toro Negro River
Catchments of 10 to 300 square miles	11	Germany	River Queiss
	12	England	Derwent Valley, Yorks
	13	United States	Cane Creek, N. C.
	14	"	Elk Horn Creek, W. Virginia
	15	"	Rio Grande, Texas
	16	South Africa	Near Cape Town
	17	"	Near Port Elizabeth
	18	N. S. Wales	South-East Districts
	19	India	Near Upper Jhelum Canal
	20	"	"
	21	"	Burma, Thapaingaing Aqueduct
	22	"	Near Upper Jhelum Canal
Catchments of 400 to 1,750 square miles	23	N. S. Wales	South-East Districts
	24	Mexico	Santa Catarina River, Monterrey.
	25	United States	Black River
	26	"	"
	27	"	"
	28	"	Catawba River, N. C.
	29	"	Arkansas River
	30	India	Ondal-Sainthia Railway
	31	Germany	River Oder
Catchments of 2,000 to 10,000 square miles	32	United States	Catawba River, N. C.
	33	Canada	S. W. Ontario
	34	Queensland	Brisbane River
	35	"	"
	36	India	Kali Nadi River
	37	"	Penner River, Madras
	38	"	"
	39	"	"

At long intervals, once perhaps in fifty or a hundred years, a flood is liable to occur which far exceeds all previous records. The intensity, duration and distribution of the rainfall are such as to cause the various streams to bring down maximum floods and to bring them simultaneously. Such a flood is

Catchment Area (Square Miles)	Flood Discharge per Square mile of Catchment Area (C ft. per second)	Remarks
6	4,000	
.88	4,165	
1 47	3,825	
2.96	2,214	
3.87	3,744	
5 to 10	1,613	Revised estimate
6 6	480	
91	135	
2 5	84	
3 33	3,200	
12 35	358	
14 6	311	
22	1,341	
44	1,363	
85	412	
34 5	78	
35	640	Estimated
49	37	
48 5	1,312	
56 45	1,000	Revised estimate
172	327	
174	550	"
418	11 2	
544	590	
729	31 5	
950	30	
1,350	30	
1,535	62	
1,740	57	
1,350	141	
1,640	46	
2,987	50	
2,600	196	
5,200	43	
5,200	45	
2,593	51 or more	Roughly estimated
2,100	82	Rugged uplands
7,000	23	"
10,000	17.7	"

not likely to occur but it may occur. It may occur very soon as in the case of the Kali Nadi. Generally speaking the expense of allowing for it is — at least in the case of a large catchment area — too great. Regard however must be had to the amount of harm which is likely to occur from failure of any work to deal with the flood.

Further remarks as to the investigation of flood discharges are given below under Rain Storms.

It has been stated that the larger the catchment the less, generally, is the flood discharge per square mile. This can readily be seen from the accompanying table [1], especially if the figures of any one country are selected. The cases discussed above are included The highest figures are of the most interest The discharge of 590 cubic feet per second per square mile from the area of 544 square miles in Mexico is remarkable It is said to have far surpassed all other records known to the writer [2] Some of the figures in the table are commented on below. The Upper Jhelum catchments are in the Pabbi hills Their slopes are steep and cut into ravines by the rain. This favours rapid run-off. There are scattered trees and bushes Such details as to the other catchments as are available and have not already been given are shown in the remarks column "Revised estimate" indicates that the figures were re-estimated after some experience of the catchments

Records of properly observed flood discharges are scarce, though more trouble is now being taken to obtain and publish them

In the case of the Derwent valley area a smaller flood than that shown in the table was also observed and the maximum rate of discharge — 83.6 c.ft per second — was 1.18 times that of the average rate of the rainfall.

In Great Britain the flood discharges of rivers do not exceed 13 c.ft. per sec per square mile of catchment. In

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[1] See Reports quoted above Also *Trans Am Soc C E*, Vol 47, pp 445—454 Vol 48, pp 1092—1247, *Proc Inst C E*, Vol CLXXI p 360 and Vol. CXCIV, pp. 3—152, *Engineering News*, 2 Nov and 9 Dec. 1911 and 18 June 1914 and *Irrigation Works in India and Egypt*, Buckley.

[2] *Engineering News*, 23rd Sept 1909.

mountainous districts in the North of England and in Scotland the flood discharges per square mile of small catchment areas have been found to vary from 64 to 320 c.ft. per second.

Time-factor in Flood Discharges. — Generally in the case of a flood the chief figures required are the probable highest water-level and the maximum rate of discharge Q, of the flood, per second. But if the flood water is to be stored the estimated time T, from the commencement of the rise to the end of the fall is also required so that the volume can be calculated Rough hydrographs of floods have been given above (Fig. 98, p. 196) The total volume discharged may be about $\frac{QT}{2}$.

In case No. 15 in the table the rainfall on the catchment area was 12.5 inches in 30 hours. The average rate per hour was thus .417 inch This gives 267 cubic feet per second per square mile. For the 85 square miles the quantity of rain water is 22,667 cubic feet per second for 30 hours. The time occupied by the passage of the flood in the river was 6 hours, the discharge during that time increasing from 4,000 to 35,000 cubic feet per second and then decreasing again to 4,000 cubic feet per second The average discharge would be perhaps 19,500 cubic feet per second for 6 hours. This is only about 17 per cent of the rainfall without any deduction for the discharge which was passing down the stream before the flood and continued to pass down after the flood was over [1]. The discharge per square mile of catchment was $\frac{19{,}500}{85}$ or 229 cubic feet per second per square mile. But the maximum discharge was 412 cubic feet per second per square mile. At a place somewhat lower down the stream the discharge, during a period of about 11 hours, increased from 4,000 to 23,700 cubic feet per second and fell again to 4,000 cubic feet per second. This gives a much smaller maximum figure. The catchment area was probably the

[1] The flood figure gives the whole discharge and this procedure is correct, because, as already stated, the engineer requires the maximum figure with which he will have to deal

same or only slightly greater. This case shows that even in a fairly large catchment the rate of the flood discharge when at its height may far exceed the average rate at which the rain water is falling on the catchment. The distribution of the rainfall may have been such as to cause a sudden coming together of streams.

In the above case full information is given. In some others the figures supplied merely show the average discharge while the flood lasted. This remark applies to the Queensland catchment — cases Nos 34 and 35 — and it may possibly apply to cases 8, 9, 16, 17, 18 and 23. Presumably the discharges refer in every case to a point near to the exit of the stream from the catchment area. Complete information is required in all cases in order that proper deductions may be made.

In the case of the Queensland areas the rainfall on the catchment No. 34 was 15 inches in 8 days and 64 per cent. of it ran off; on No. 35 it was 9.77 inches in 5 days and 85 per cent ran off.

Rain Storms. In most cases a flood is caused by a particular rain storm. A rain storm may have any diameter from a mile or two upwards. The rainfall is most intense at about the centre of the storm In a storm over London which lasted 2½ hours the intensities of the rainfall were as follows. [1]

Area (sq. miles)	.4	4	15 4	31
Rainfall (inches)	more than 4	3 to 4	2 to 3	1 to 2

A rain storm at any place may last for 1 day or more. It hardly ever lasts for more than 5 or 6 days Of course the quantity of rainfall is not in proportion to the length of the storm. The length of storm which is most likely to cause a heavy flood is a matter of local experience The time is probably greater the larger the catchment area. On a small catchment such as a municipal area it is generally sufficient to consider the heaviest known fall in 6 hours. Such a fall will be one of great intensity and any longer fall is not at all likely to cause a heavier discharge.

[1] British Rainfall, 1917.

The centre of the rain storm may of course be outside the catchment. Ordinarily the rain falling outside the catchment receives no attention except that rain gauges near the catchment may be read in order to estimate the rainfall at its edge. Let the falls during a rain-storm be observed not only on the catchment but on the whole area covered by the storm and let isohyetal lines be drawn, over the whole area, for the 1-inch, 2-inch &c. falls, and the necessary calculations of areas be effected. Then time-depth-area curves (Fig. 100) can be drawn for the whole storm area. By assuming that the centre of the storm has been shifted to the centre of the catchment it is easy to find from the diagram what the rainfall on the catchment would in this case have been.

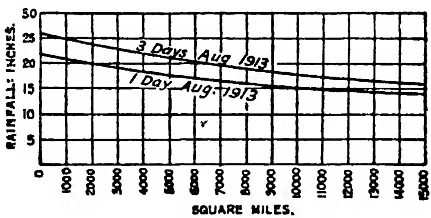

FIG. 100

This procedure is a distinct advance on the usual method. It was adopted in the case of the great flood in the Miami Valley, Ohio (p 228). All the rainfall records of the eastern part of the United States were examined. Iso-pluvial lines for the 1-day, 2-day &c , falls were drawn. The conclusion arrived at by the engineers was that the greatest floods were caused by rain storms of about 3 days. The flood discharge to be dealt with was arrived at by allowing for a total rainfall of 10 inches in 3 days for the smaller catchments on the tributary streams and 9 5 inches for the larger catchments.

The figures for the Miami storm were exceeded by only a very few and were found to be exceptionally high if the Ohio region alone were considered. It was judged that there was no likelihood of their ever being greatly exceeded but

the American rainfall records are somewhat sparse and it was decided to provide for a rainfall 40 per cent. in excess of what had occurred in 1913.

Flood Discharge Formulae. — The tendency for the flood discharge per square mile of catchment area to decrease as the area increases has long been known and attempts have been made to found on it formulae for calculating flood discharges. The formulae are often of the type

$$Q = cM^n \quad \ldots \ldots \ldots \ldots \ldots \quad (3)$$

where Q is the flood discharge in cubic feet per second per square mile, M is the area of the catchment in square miles and c and n are constants. Most of the formulae tend to give over-estimates of the flood discharge. In the original formula by Dickens, used in Bengal, n is .75 and c is 825. In Madras n is .67 and c varies from 363 to 675 in different kinds of country. When the figures for a given catchment are known the formula can be used for obtaining rough figures for another catchment of not very dissimilar character as regards topography, soil, climate and rainfall. But for other cases the constants may vary very greatly. Formulae of the above type are considered by some writers to be useless. They are of some value as giving rough preliminary approximations in cases where no observations have been made.

Far better than any of the other older formulae is that of Craig,

$$S = 184 \text{ B} \log \frac{8L^2}{B}$$

This gives, not the discharge, but the cross-sectional area S, of the flood water. L and B are the length and breadth of the catchment. The velocity has to be calculated separately.

A stream of course keeps varying in slope and section This need not introduce confusion into the case. The general slope is alone considered. It can be imagined that the stream has been regularised and graded. If the slope were such as to give a velocity of 1 foot per second the formula would give the discharge from the catchment. As a preliminary the velocity can be estimated without calculating it from

the slope. The velocity in a small stream with a steep slope does not differ greatly from that in a large stream with a flat slope. It is generally from 2 to 8 feet per second.

Art. 3. Flood Embankments. *Alignment and Height.* A flood embankment may be close to the edge of the river or it may be set back. If set back it need not follow all the windings of the channel. The setting back of an embankment gives an increased waterway to the stream during floods, and therefore a lower flood level, but the effect of this is slight in cases where the depth of the water on the flooded land is small, especially if such land is covered with vegetation, or is otherwise much obstructed. Setting back is necessary in cases where the stream is liable to erode the banks to any considerable extent. In such a case the embankment should not be so near to the river as to be in much danger from erosion, but the ground, as already stated, generally falls in going away from the river, so that when an embankment is set well back it is in lower ground, more expensive and more liable to breach. The most suitable alignment is a matter of judgement and depends largely on the extent to which the river is likely to shift.

Embankments should, where possible, be made in straight or properly curved reaches. A flood embankment, at least at its upstream end, should terminate in ground which is above flood level. The top of an embankment should be, in the case of a large river, about 3 feet above the high flood level of the river. It should, of course, be graded parallel to the general high flood level, but neither the gradient nor the height of the flood may be known with accuracy (Chap. I., Art. 2). There is generally a record or mark of some high flood and this is taken provisionally as the flood level. Or the level is calculated approximately from the flood readings on the nearest river gauge. If experience shows that the embankment is too low, it is raised. Enquiry should be made as to whether the stream has any special tendency, such as that which occurred on the Irrawaddy (p. 7), to alter its level at one place with reference to that at a place higher up.

When an embankment is near to the river the flood water

may flow along it with considerable velocity and the slope may have to be protected by fascining or by any of the methods described in Chapter IV., Art. 3. When the embankment is set far back the flow along it is not, in most cases, rapid but owing to the large area of the flood water the wind sweep is greater and protection against waves is likely to be necessary. This remark applies also to the bank of a canal which may be near a river. Such a bank often acts as a flood embankment. It is not necessarily in any particular danger owing to its being in contact with flood water.

Where the country bordering on the sea is below high water level, as in the case of the English fen country, there

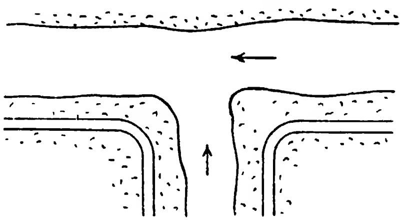

Fig. 101.

are embankments parallel to the coast. When an embanked fenland river (Art. 5) enters the sea at an acute angle with the coast line the river bank forms part of the sea embankment. The banks of the fenland rivers are often of great strength where the rivers have been straightened and much earth was consequently available, the slope on the river side being made very flat and the top being say 6 feet or more above high tide level.

Subsidiary and Connected Works. — Where an affluent enters the river it will probably be necessary to run out branch embankments (Fig. 101) so as to prevent the affluent from flooding when it is held up by flood in the main river. The branch embankments run back till high ground is reached. Sometimes cross embankments are

run from the main embankment to high land. Their object is to localise the damage if a breach occurs. Along the back of the embankment there may be a drain. There may be sluices or regulators in the embankment for the purpose of allowing water to pass through from the river side for purposes of irrigation, or from the land side for drainage when the river is low (Art. 5). Sometimes the drainage water is pumped temporarily into a reservoir. A drain may be carried, by a syphon, under another which has started higher up [1].

In order to avoid the necessity for raising flood embankments, or to reduce the strain on them, waste weirs — "spillways" — or regulators are sometimes provided in the embankments, the water escaping into low lands and flowing parallel to the river — or by a separate river mouth — into the sea. At New Orleans proposals have been made for a new spillway into Lake Borgne [2] (see also p. 228).

Sometimes when a main embankment is set far back, a subsidiary embankment of smaller section is constructed closer to the stream. This is often objectionable. The smaller embankment is liable to breach, and the water then rises suddenly instead of gradually against the main embankment, which is thus endangered to some extent, especially as it is dry instead of being partly soaked.

Along the great shifting rivers of the Punjab there are long lengths of flood embankments. They are constantly crossed by the inundation canals. At the crossing there is generally a regulator but it is sometimes omitted if the ground is high and the canal has good banks upstream of the embankment. Owing to alterations and additions the systems may become complicated. These matters are dealt with in more detail elsewhere [3].

Effect on Regime of River.—If embankments are made along both sides of a river the immediate effect is likely to be a raising of the flood level. But the increased velocity of the

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[1] *Trans Am. Soc C E*, Vol 41, pp 523–547

[2] *Engineering News Record*, Vol 90, pp. 21–27.

[3] See foot-note p. 136

stream — accompanied by an increase in its rolling power and possibly of its carrying power — may cause scour of the bed or sides or both, and so after a time nullify the rise of the flood level. It may not even permit it to occur. In other cases the rise may be permanent, as for instance when the bed of the stream is too hard to be scoured. One reason for a rise in the flood level is that a river, before it is embanked, parts with silt by depositing it on the flooded land. When the flooding is stopped, the charge of silt in the stream is increased and some of it may be deposited on the bed.

In any case which may arise, enquiry must be made as to whether the river channel is tending — as is common in alluvial tracts — to rise independently of embankments, whether the flooding occurs regularly or only occasionally, what the volume of the flood water is, whether it is permanently diverted (pp. 61 and 62) or spills in the usual manner (p. 60) and flows back to the river, and to what extent silt is deposited by it. Also to what extent the yearly removal of grown crops, grass or timber from the flooded tract tends to nullify the raising of the ground level by silt deposit.

If the flooded area near the river is being steadily, even though slowly, raised by silt deposit and if the flooding continues unabated, it must — in the absence of upstream causes tending to increase the severity of floods — be inferred that the river channel is also rising. It is possible that this was the case in some instances before double embanking was effected and that the embankments, as the water-level in course of time approached nearer to their tops, merely brought the rise into prominent notice.

Double embanking has raised the flood levels of rivers in Bengal and also probably the bed levels. On the Mississippi from Cairo to the sea — some 1,150 miles — it has raised the flood level but not the bed level. Double embanking extending over long lengths of the Indus and Chenab is not known to have caused rise in the flood levels but the embankments are set very far back — on the Indus they are often 6 miles apart — and the floods do not always come up to them. On the Theiss the flood level has been raised

but great numbers of cut-offs were made besides the double embanking (see below). On the Garonne and Rhine the flood level rose but it fell again as scour took place. On the Danube it did not rise.

A rise in flood level, following double embanking, increases both D and V and may alter their ratio, but it is unfortunately not known in which direction the ratio is likely to be altered. It has been suggested by Samuelson[1] that double embanking can safely be undertaken if a river scours its bed in floods. This seems to be correct for most cases. It is at least known that the rolling power is increased in floods. He also states that if a river silts in floods, embankments would probably increase the evil. This does not seem to be correct. The rolling power would be increased.

Cases in which a permanent rise in the flood level can be directly and clearly attributed to double embanking alone are not common. They are generally — as in the instances of the lower Mississippi and of the Bengal rivers — cases where the slope is flat and the velocity low. This may be because the rolling power of such a stream is small.

When an embankment is made on only one bank of a river while flooding can take place on the other bank, the chances of a rise in the flood level, or in the channel, are small.

It is sometimes said that one effect of double embanking a reach of a river is to increase the severity of floods further downstream. The importance of this may be exaggerated. The narrowing of the flood stream in the embanked portion causes the flood to rise higher and to travel more quickly in that particular reach. At a place immediately downstream the same effect is produced — but in a much less degree — because the flood wave arrives in a less flattened form. There may however in some cases, be a rise in the flood level downstream of the embanked reach, due to the increase in the discharge of the stream. The absorption and evaporation are less than before owing to the reduced area of flooding in the embanked reach.

[1] *Note on the Irrawaddy River*, Government Press, Rangoon.

Construction and Other Details. — The cross-section of an embankment depends on the soil and on the extent of damage which results if a breach occurs.

An embankment may suitably have side slopes of 4 to 1 on the river side and 3 to 1 on the land side, with a top 10 feet wide and 3 feet above high flood level. On the Irrawaddy the top width is generally 8 feet. For very high and very low embankments it is 10 feet and 3 feet respectively. In Holland, Germany, Italy and Austria the top width of embankments is often about 15 to 25 feet but the side slopes are somewhat steep.

On the Mississippi the top is 6 to 10 feet wide — with a transverse slope of 1 in 10 or 1 in 20 — and 3 feet above high flood level, the slopes being 3 to 1; or 3 to 1 and 4 or 5 to 1. In some places a berm or "banquette" (Fig. 102) is

FIG 102

added on the landward side. Such berms are also used in other countries. The added earth is all below high flood level and therefore all useful, and more can readily be added in time of danger, but a flattening of the whole slope, as shown by the dotted line, is often preferable.

Earthwork in banks has been dealt with on p. 140, soils and trial pits on p. 130, the hydraulic gradient on p. 183. Embankments require to be made with great care. The earth should be deposited in layers. In Holland, horses are driven up and down over each layer. In some parts of India the earth for embankments is brought from the borrow pits by scoops drawn by bullocks The earthwork is of so excellent a character, owing to the earth being trodden down, that no settlement has to be allowed for. On the Mississippi embankments or "levees" similar scoops have been used and drawn by mules [1] The remarks made on pp. 185—186 as to outside borrow-pits are applicable. Borrow-pits made on

[1] *Notes on the Mississippi River*, E. F. Dawson.

the river side of an embankment may silt up quickly if the river is near.

Where the soil is sand the top and face of the embankment should be of good stiff soil, if it can be obtained, for a thickness of 9 inches or a foot or else the face next the river should be protected by fascining for 2 feet above, and several feet below high flood level. Sand, protected as above, makes a good embankment, and rats do not burrow into it. Of course, if a breach occurs in an embankment consisting mainly of sand it may enlarge very quickly. Recent experiments with sand show that it tends to settle until a minimum volume is approached. Dry sand recently deposited occupies the most space and settles gradually. Wet sand settles quickly and occupies the least volume. Sand embankments under construction should be frequently watered [1]. In some cases an embankment has a core wall of sand or of clay puddle As to core walls and the use of selected earth for particular parts of the cross-section see Chap. IX, Art. 3. In Holland, on sandy soil, a trench 8 feet wide is made and taken down to the clay.

Protection of the slope may be necessary in any case where waves are liable to occur The wave action on the Mississippi embankments is heavy. As a protection against it Bermuda grass is cultivated on the slopes In specially exposed situations plank revetment is used, and in very high floods, temporary protection by means of bags of earth, planks or bundles of sugar-cane or cotton refuse. In Holland embankments are turfed, and trees and shrubs are not allowed to grow In the Punjab the growth of all kinds of jungle is encouraged, at least on the water slope. In Bombay, jungle is kept down. It serves as a cover for rats which otherwise would be more likely to be destroyed by birds of prey, but it binds the soil together and protects it from the wash of waves, and from winds which blow away sand and dust and may so wear the embankment slowly away.

In embanking a long reach of a river it is convenient to begin from the upstream end, because otherwise floods may

[1] *De Ingenieur*, Vol 34.

get behind the finished part of the embankment and, becoming impounded in a "pocket" formed by the embankment and high land, rise to an abnormal height and, unless gaps in the embankments have been left or are subsequently made, cause breaches and do much damage

During high floods pegs should be driven in at frequent intervals, to mark the high flood levels. The levels of the pegs can be observed at leisure.

The River Theiss. — The Theiss flows through the plains of Hungary and has a fall of only .014 ft. to 0018 ft. per 1,000 feet. The length of the valley is 440 miles and a very great part of this used to be flooded for a width of 7 or 8 miles and for a great part of the year. The flood discharge at the debouchure into the Danube is some 140,000 c ft. per second.

The river was studied for 10 years, commencing in 1830, by a very large staff of engineers. The remedial measures proposed were the construction of bold cut-offs and of flood embankments on both banks, the width between the embankments ranging from 650 to 3,300 yards. A rise in the flood level due to the embanking was foreseen and it was arranged that this should be watched and allowed for in the construction so far as might be found suitable. Many of the cut-offs were to be deferred till the effect of the embanking suggested the best places for them Some narrow reaches in the river were to be widened and mouths of some tributaries corrected (p. 63). At Szegedin the width was to be increased from 170 to 250 yards, the embankments were to be 800 yards apart and the mouth of the Maros which enters at a bad angle above the town was to be shifted to below the town

The works were carried on from 1849 to 1867, and 4,200 out of the 6,000 square miles of country were protected from floods. Political and other changes then caused the abrupt cessation of the work. After that, floods now and then broke into the protected area but the damaged area kept decreasing. In 1879 the town of Szegedin was flooded — and much of it destroyed — because the municipality had declined to take their share in the works and the proposed

local widening of the Theiss and diversion of the Maros had not been carried out [1].

Maintenance. — The closure of breaches is dealt with in Chap. IV., Art. 2. When a breach occurs the first thing to do is to protect the ends so that the breach shall not lengthen. In China on the Yellow River breaches are closed by means of millet stalks 6 feet long, made into bundles with straw ropes and packed with stones and sand The final closure is effected by a thick raft of such bundles held by more than 100 straw ropes each 3 inches in diameter. A bank of earth and stones is tipped on each side and rip-rapped [2] On the embankments in the east of England breaches are sometimes closed by filling barges with clay, sinking them in the gap and adding loose chalk and bags filled with clay.

If the water which passes through a breach in a flood embankment becomes pocketed by another bank running obliquely or by high ground, it may be necessary to cut the flood embankment to let the water flow back towards the river.

The stopping of leakages and general maintenance of banks are dealt with on pp. 191--192.

Art. 4. Other Methods of Flood Prevention. Next to the construction of embankments the most usual and effective method of dealing with flood water is to increase the discharging capacity of the stream and so to lower its water-level. Another method is to construct reservoirs for storing flood water.

Lowering the Water-Level — The water-level of a given length of stream can be lowered by lowering the bed, widening the channel or straightening the channel The efficiency of these processes is generally in the order named. The alteration to the channel must in any case be continued to some point a long distance downstream of the reach under consideration. Let the channel be supposed to be of "shallow" section with sloping sides. Let W be the mean width, D the depth, and S the slope. Let it be required to lower the water-level

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[1] *Proc. Inst. C E.*, Vol LIX p. 381

[2] *Trans Am. Soc C E*, Vol 48, pp. 1092—1247

by an amount equal to $\frac{D}{5}$. This can be effected by lowering the bed by about 25 per cent. of D; or by increasing the width by about 50 per cent, or the slope by about 100 per cent. If the bed is lowered, V is not affected and the mean width is slightly reduced. Increase in W reduces D and therefore reduces the hydraulic radius and the velocity. Hence the large amount of widening necessary. When S is increased, the velocity, if R remains the same, is affected only as $\sqrt{S}$, but the depth of water is reduced and R therefore reduced.

It does not, of course, follow that lowering the bed is always the best plan and straightening the worst. Any one of the processes may be more or less impracticable because, for instance, of the hardness of the material to be removed. If a channel is very tortuous, straightening it may be the easiest and best procedure. In soft soils one advantage of straightening is that diversions can be dug to a small section, and left to enlarge themselves (p. 103).

A particular kind of widening consists in digging a new channel and keeping both the new and the old channel open.

If a channel contains a weir, or a local raised portion of bed forming a kind of submerged weir, or a contracted place or narrow bridge, the upstream water-level can be lowered by simply removing or reducing the obstruction The lowering of the water-level will be greatest at the site of the obstruction, and will be zero at some point far upstream. If the raised portion forms a long shoal, its removal — supposing its height above the general bed to be the same — will have more effect than if it were short If the height of the raised portion is small compared to the depth of water, or the amount of contraction small compared to the width of the stream, the removal may have much less effect than might appear likely In the case of the narrow bridge it is often feasible, instead of pulling it down, to lower the bed and add a low-level floor which slopes gradually, upstream and downstream, up to the bed level, the sides being pitched.

Clearing away vegetation, overhanging branches etc.

from the sides of a channel, removing obstructions in the bed or dressing the sides, each have the same effect as making it smoother, and are equivalent to a slight widening. Any of the alterations mentioned in this and the preceding paragraph may often be effected at a small cost compared to the benefit obtained.

It is of course well-known that in a silt-bearing stream there is a tendency for silt to deposit where the sectional area is greatest. It has been stated (p. 58) that there is some tendency for silt to deposit in a deep place even if the sectional area is not in excess. This is because the ratio of D to V is great and also because heavy rolled material tends to accumulate in low places. Such places, however, do not always silt. They abound in natural streams and also in artificial streams where contractions occur The question whether such a deep place will or will not silt depends on the kind of silt and solids usually brought down, on the ratio of D to V and — as regards rolled material — on V. If the bed at a narrow bridge is lowered it does not follow that any silt will be deposited there. In cases such as the above the tendency to silt is greatest at low water, when D at the deep place may be very much greater than elsewhere If the channel is used for passing off flood water, the low water of a falling flood — if highly charged — may deposit silt in deep places though the next flood as it rises will probably scour it out again.

In the case of the works — mentioned again below — for mitigating floods in the Miami Valley, increased width of channel was limited because of adjacent streets and buildings. But it appears that extra deepening in narrow places was avoided because the bed gradient would not have been uniform [1]. The procedure may have been correct — the full details are not known — but the reason given for it appears to be inadequate The narrow places may perhaps, of themselves assume lower bed levels and so do their full work.

A Series of Cut-Offs. — The effect of a cut-off is shewn in Fig. 39, p. 92. A scour wedge is formed upstream and a silt

[1] *Engineering News Record*, Vol 85, p. 292

wedge downstream, the net result being a locally steepened slope shown by the dotted lines. There may also be falling in of the banks in the steep reach and widening of the channel. If the original channel was only just stable the steep slope becomes in time flatter. Stability is attained when it becomes nearly as flat at the general slope of the channel. The flood level upstream is lowered but that downstream is raised.

If, in a fairly hard and stable channel AB (Fig. 103) a series of cut-offs, C, D, etc., are arranged so that the silting of the first one extends down to about the point where the scour of the second one begins, the slope of the whole channel is steepened and becomes H L, the point A shifting to H because of the reduced length. It may be that the silt from

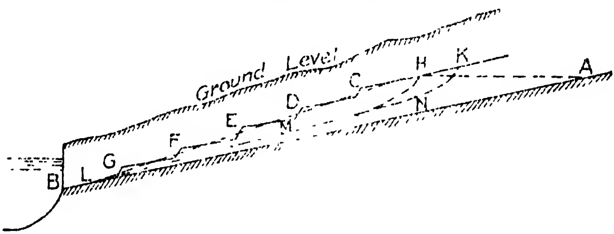


Fig. 103

the lowest cut can be allowed to remain in the channel L B, or will not deposit there.

In a prolonged flood the river channel acts as a reservoir. Cut-offs reduce its capacity only slightly. They increase its capacity until the loops have silted up

The effect of a series of cut-offs in a river which is only just stable needs somewhat careful consideration. The question was recently raised by an eminent engineer whether, in such a river, cut-offs can be made to such an extent as to permanently lower the flood level — possibly bringing it within soil — and still leave the channel stable, while avoiding any other heavy expense.

Suppose that cut-offs have been made as in the case last considered and that the channel has assumed the bed shown by the dotted line HL. There is then steady scour of the

bed due to both the increase in V and the decrease in D. The bed will be lowered — the channel near H being assumed to be hard — somewhat as shown by the dotted line HML or KNL.

There will be a tendency, due to the increase in V, to scour the banks in all reaches, straight or curved. For a time at least the tortuosity of the channel will not increase appreciably, especially as some of the worst bends will have been cut out. The bank can be protected where necessary. The slope is steepest in the upper part of the channel while the water is taking up its additional charge of silt. After that the tendency to scour is confined to the banks. The lowering of the bed of the channel, however, increases the height of the banks, so that the material brought into the stream by a given amount of widening, is increased and the stream is less able to carry it away Also the flood water formerly spilt over the banks and deposited silt on them. The cessation of this action — when the stream becomes within soil — increases the silt charge and this tends to reduce the scour.

There seems to be no reason why a general lowering of the water-level on the above lines should not be feasible If AL is 500,000 feet, the slope being 1 in 20,000, and if the channel is shortened by 1 mile in every 10, the length AH is 50,000 feet. Scour at H would have to be only 2 5 feet in order to restore the original bed slope Bank procection — more or less — would be required. There would be difficulty — in spite of the most detailed investigations — in estimating beforehand the ratio in which the length of the channel would ultimately have to be reduced. The best sites for cut-offs having been selected, some of them would be made. Their effect would be watched, the others being held over. The most difficult period would be that in which the bed was being lowered to HML It would probably be necessary to open the lower cut-offs first. The loops of the old channel would be available as receptacles for silt.

If, as is likely, the slope of the river diminishes in going downstream, that of the shortened channel would also diminish. The ratio of the length cut off in any long reach to

the total length of the reach could be about the same throughout.

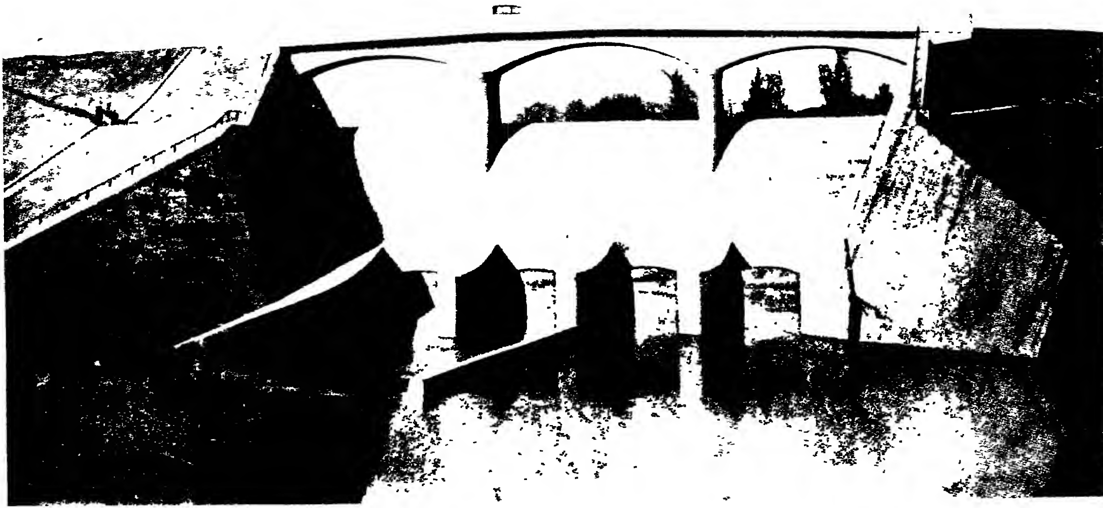
It has been said that in the lower reaches of the Mississippi near New Orleans, where the slope is extremely flat, cut-offs are useless as a remedy for flooding. They would however have the same relative effect as any other cut-offs in increasing the gradient. They would tend to lower the bed. They would be ineffective very near to the sea. An alternative would be to improve the various mouths of the river (see also p. 217).

If the river, instead of having average stability, is one of those which tend to deteriorate and has a very low velocity, for instance any of the English fenland rivers, there would be no necessity for keeping any of the cut-offs in reserve.

Reservoirs. — One method of mitigating or preventing floods is the construction of reservoirs for storing the water. Reservoirs locally known as "washes", formed by setting back the embankments, exist on the English fenland rivers. One wash, on the Nene, below Peterborough, is 12 miles long and half a mile wide and is filled, in floods, to a depth of 7 feet and holds 1 inch of rainfall over the river basin, and this is found to be sufficient Reservoir construction in England is however in most cases impracticable owing to the expense. To store the water which is given by 1 inch of rain in the basin of the Thames, a reservoir would be needed 50 feet deep and covering about 7 square miles. It might cost £ 15,000,000

Flood reservoirs are not suitable in cases where the flood water transports much heavy silt and solids.

In 1913 a highly disastrous flood occured in the Miami River, Ohio. The river flows for 120 miles through rolling country abounding in short steep slopes, and the soil — a mixture of clay, sand and gravel — is somewhat impervious There are also many drains for fields and alongside of highways and railways. The conditions favoured a high ratio of run-off. The run-off in some parts of the catchment area varied from 73 to 91 per cent, of the rainfall, the higher figure however being in a town area.



Taylorsville outlet works, Miami River

January 1913 was very wet, February rather dry. In March when the ground was wet, there was heavy rain on four consecutive days, the total being 6 inches to 11 inches. The flood discharge was some 250,000 cubic feet per second which was far in excess of the capacity of the river channel. There was great loss of life and immense destruction of property in the towns through or close to which the river flows.

The river valley is flat and ½ mile to 3 miles wide and is below the level of the surrounding rolling country. There are thus unusually favourable sites for storage reservoirs or "retarding basins". Flood embankments seem to have been impracticable because of the towns. The scheme of flood protection consisted of widening and improving the river channel and the construction of 5 retarding basins. These are formed by earthen dams thrown across the valley. The water is passed through the dams by means of masonry arched openings or "conduits" which are only capable of discharging as much water as can be safely dealt with by the improved river channels downstream of them. The latter are intended to dispose of ⅓rd of a maximum flood, the reservoirs ⅔rds.

The flood discharge to be dealt with was arrived at by allowing for a total rainfall of 10 inches in 3 days for the smaller catchments on the tributary streams and 9.5 inches for the larger catchments.

The crowns of the arched conduits are submerged during floods and the flow through them is like that through an orifice. They are slightly bell-mouthed. This arrangement was preferable to a weir and is sound in principle. It gives a smaller discharge when the flood is at its height.

In the earthen embankment there is a gap which is occupied by a concrete overfall dam. The ends of the embankment are held up by massive retaining walls. The overfall only comes into use in extreme floods.

In two of the reservoirs the conduits pass under the earthen embankment. In the other reservoirs the overfall dam is pierced by the conduits. In high floods the velocity through

the conduits will be 50 or 60 feet per second. The foundations are on rock.

The two conduits, side by side, discharge into a gradually widening concrete channel (Fig. 104) whose floor curves downwards and becomes a stepped and roughened incline leading into a deep pool confined between the side walls. The object of this is to cause a standing wave and dissipate the energy of the water. Below the pool there are two weirs across the channel. These regulate the transverse distribution

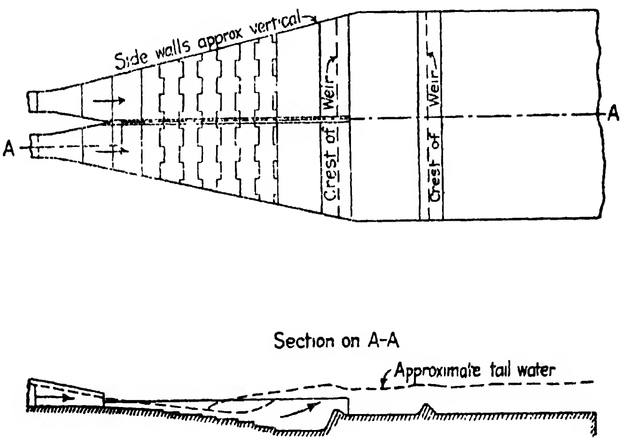


Fig 104

of the out-flowing water and prevent its concentration at any part of the channel with irregularity in the standing wave.

Exhaustive small scale experiments were made with standing waves. The arrangement adopted was found to be the best. The roughening of the floor diminishes the velocity, leaves less work for the "jump" to do and increases its stability [1]

The standing wave or jump is the best means of dissipating the energy of a large mass of water. The conditions necessary

[1] State of Ohio Miami Conservancy District. *Technical Reports*, Part. III. Dayton, Ohio, 1917

for the existence of a complete standing wave are given in *Hydraulics*, Chap. VII. Art. 11.

The Grand River in South-western Ontario has a maximum flood discharge of 51,000 c.ft. per second. This lasts for only a few hours and the flood for at most 2 days. The floods do much damage to agricultural land. It is estimated that if 14,000 or 15,000 c.ft. per second could be impounded for 2 days the floods would no longer be destructive. The necessary reservoir capacity would be about 2,600 million cubic feet. A suitable site has been found. Such a reservoir would give a well-sustained flow during the summer and would benefit various towns and water-power schemes [1].

Storage of flood water cannot well be combined with storage for power, unless the reservoir is of excessive size, because the space where the flood water is to be stored must be kept empty. Sometimes reservoirs for irrigation works are utilised for flood storage but this is not often practicable or convenient.

Art. 5. Drainage of Flat Lands. In a tract of country which is not flat the capacity of the natural streams for carrying off the rain water is generally sufficient, except for the occasional flooding of small areas near to their banks. But in any area of flat land the subsoil water-level may be too high — having regard to the requirements of agriculture, health or other matters — being perhaps only a short distance below the ground level. It may even be above it so that the area is a swamp. When it is within three feet of the surface the land is said to be "awash". Unsatisfactory conditions of this kind may exist all the year round or they may exist only when the rivers are high In order to lower the water-level the area can be intersected with drains. The smaller or "branch" drains must have an outfall into a main or "arterial" drain.

When the area of flat land is great, the main or arterial drains are the rivers, or large artificial drains made to supplement the rivers. In the East of England the rivers and arterial drains are roughly 6 to 10 miles apart.

When the discharging capacity of the arterial drains is

[1] *Proc. Inst. C. E.*, Vol. CCII. p. 276 (Breithaupt)

insufficient the remedy is to improve them by lowering the water-level (Art. 4). In the English fenlands — including the Isle of Ely and parts of Norfolk, Bedford, Huntingdon, and Cambridge — the conditions have become distinctly bad. The neglect is largely due to the multiplicity of authorities concerned with any one river.

For any area not very small, a contour survey is required in order that the number and alignment of the branch drains may be fixed with reference to the ground levels and that the best outfall may be decided on. If the outfall is into a stream which has an appreciable surface slope, a lower

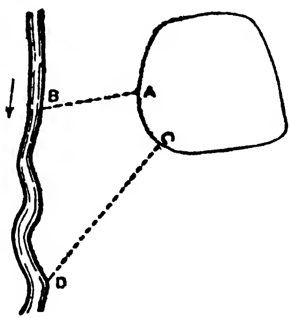


FIG 105

outfall can be obtained by constructing the drain on a line such as CD (Fig. 105) instead of on AB. Enquiry is also necessary as to the nature of the sub-soil If it is all pervious, matters are simplified, but if there is clay or other impervious material, its extent and position must be ascertained and the drains so arranged that its obstruction to the flow of sub-soil water into them shall be a minimum. In order to avoid needless interference with property the branch drains should, as far as possible, run alongside roads or along the boundaries of estates

When no outfall at a low enough level can be found, recourse is had to pumping. The branch drains can discharge into a tank or pit from which the water is pumped by steam

engines or wind-mills and is discharged into a drain at a higher level.

The discharging capacities of the drains must be such that they will carry off the yield or run-off of the area during heavy falls of rain in short periods, say 24 hours. Thus the sub-soil water-level, once it has been lowered, cannot be raised again.

In India and in places where, owing to canal irrigation, the water table is within about 10 feet of the ground, 30 inches of rain in the monsoon may bring it up almost to ground level (p. 21). Sir John Benton has pointed out that this agrees with calculations since the soil has 25 per cent. of voids in it. To prevent the water rising, drains 150 to 200 feet apart and 3 or 4 feet deep are ineffective. They must be closer together and much deeper [1]. The same difficulty has occurred in Egypt. The only real remedy in such cases is to line the irrigation channels (p. 189).

Except during rainy weather the discharges of the branch drains are generally imperceptible or at least extremely small. The drains may be practically dry at most seasons, the water from the soil percolating into them slowly and being evaporated. The bed level of the drain must be low enough to drain the adjoining soil to the requisite depth or to as low a level as is practicable, having regard to the conditions, at the outfall. In England 3 feet is the minimum depth for the water-level of the drain below the ground. The gradient need probably be only an inch or two in a mile. If the bed level has to be several feet below the ground level the bed width can probably be only a foot or two and the side-slopes 1 to 1. If the bed has to be higher its width can be greater. A section arrived at in this manner will probably have the necessary capacity for disposing of the run-off during heavy falls of rain. If not the capacity or number of the drains can be increased. The number of the drains at first made is to a great extent a matter of judgment. In the English fen country in a district near the mouth of a tidal river where the sub-soil is porous, the drains are, on the

[1] *Eroc. Inst. C. P.*, Vol CCXII pp 82 and 95.

average, ½ a mile apart and about 7 feet deep. They are nearly always dry. The water table is below the bottom of the drains and perhaps at about the mean water level of the river. The annual rainfall is some 30 inches, spread over the year. With a monsoon rainfall matters would be very different.

In boggy land a drain may tend to become filled up by horizontal movement of the bog. In such a case a drain is more or less an experiment and its size should not at first be great. In draining extensive swamps dredging machinery may be used.

It may of course be necessary to enquire into the question whether any underground water comes in from another watershed When the area of flat land which is to be drained adjoins higher ground, which slopes down towards it, a "catch-water drain" can be constructed, approximately along the toe of the slope, so that the drainage of the higher ground is intercepted by it The catchwater drain can be at a level high enough to enable it to discharge by ordinary flow, though the drainage water of the low land may have to be pumped.

In the fenlands in the East of England the ground level is below mean sea-level and it falls in going inland. When a river coming in from the uplands enters the fenland, it is embanked and its water is thus conveyed to the sea. The rivers are tidal. At low-tide the drains of the fenland discharge into the river through masonry inlets provided with sluice doors — like lock gates — which allow the water to enter the river but close when water begins to flow out of the river into the drain as the tide rises. The inlets have also — on the landward side — gates working in vertical grooves. Thus in times of drought the river water can be admitted into the drains for the benefit of cattle or agriculture, and prevented from flowing back into the river when the water-level falls At points on the river above the tidal reach, or wherever the level of low tide is too high to admit of direct flow from the drains into the river — or other arterial drains — the water is lifted by pumps.

NOTES TO CHAPTER VII.

Flood Discharges (p. 200). Experiments extending over a long period have been made at Danzig,[1] on an area of 32.8 ft. × 65.6 ft. covered with (*a*) sand, (*b*) stone sets laid in sand, (*c*) stone sets grouted.

Let K be the discharge co-efficient or ratio of discharge to rainfall, *i* the rainfall intensity in litres per second per hectare, T the duration of the rainfall in minutes and *m* a co-efficient depending on the nature of the ground. Then it was found that :—

$$K = mi^{x}T^{y}.$$

For (*a*), (*b*) and (*c*) respectively *m* was .0064, .0214 and .0238 ; ^{x} and ^{y} were respectively .567 and .228.

An intensity of rainfall said to be attained once a year was 125 litres per second per hectare (about 1.8 inch per hour) for 7 minutes. In this case K was for (*a*), (*b*) and (*c*), .155, .517 and .575. It was considered that the use of the above figures would lead to economy in designing drainage systems.

Upper Jhelum Catchment (p. 203). The accuracy of the discharge observation has been disputed. A recent article on the subject[2] puts the observed discharge at 3,500 c. ft. per second, and shows that this may have been not more than 76 per cent. of the rainfall on the small, soaked and egg-shaped catchment. From a catchment of 1.24 sq. miles on the same canal, a discharge of 4,607 c. ft. per second (3,715 c. ft. per second per sq. mile) was observed.

Rainstorms (p. 213). In New England a storm on 3rd to 5th November, 1927, lasted 24 to 48 hours, the isohyetals for the whole storm for 9, 8 and 6 inches giving 340, 1,200 and 7,500 sq. miles respectively. The greatest fall registered was 9.65 inches for the storm, and 7.85 inches for the highest 24-hour period.[3]

Flood Reservoirs (p. 229). In the largest of the Miami

[1] *Die Bautechnik.* Vol 7, pp. 507 and 529.
[2] *Engineering.* 7th January, 1927 (Bellasis).
[3] *Engineering News Record.* Vol. 99, p 796

reservoirs there are four outlets, each 19.2 ft. high and 15 ft. wide. The maximum flood discharge when the water is level with the crest of the spill-way will be 56,400 c. ft. per second, the velocity through the outlet being then about 47 ft. per second.

Methods of Protection from Floods (p. 223). Storage Reservoirs are often impracticable on very large rivers, unless great areas of land for them are available at small cost. In some rivers they would be liable to serious silt deposit. Afforestation, of course, takes time.

The suitability of spill-ways must depend largely on where the water will go and how far its course can be controlled. In the Sacramento valley—area 4,250 sq. miles—the protection from floods is mainly by embankments, but in these there are three large spill-ways or weirs—one movable—and the flow along the " by-passes " is restricted to definite areas.[1]

The Mississippi River Commission has decided that protection from floods on that great river must still be by embankments only. It is considered that spill-ways would cause draw-down and scour in the river, with consequent silt deposit in the river below them or in the by-passes.[2]

The effects of various large tributaries of the Mississippi have been considered, and it is stated that greater floods than that of 1927 may occur, but that their magnitude cannot be predicted until the discharges of the tributaries are observed for a number of years.[3] Remarks as to this are given on pp. 208 to 210. In 1927 there were numerous breaches in the embankments, and these would lower the flood level farther down even if the water eventually flowed back to the river.

The idea of a system of cuts-off (p. 226) originated with the late Sir John Benton. He stated that he had certain rivers in his mind, but which they were is not known. Possibly the system would be unsuited to the Mississippi.

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[1] *Proc. Am. Soc. C.E.* Vol. 53, p 2,586 (Grunsky)
[2] *Op. cit.* Vol 53, p 2,470 (Frankenfield).
[3] *Op. cit.* Vol. 53, p. 2,485 (Grover)

CHAPTER VIII

WEIRS AND OTHER STRUCTURES

Art. 1. Weirs. The best site for a weir or other permanent structure has been dealt with under Off-takes (p. 133). It should be added that if the stream is unstable and liable to shift its channel, a site immediately downstream of a bend should be avoided. This is because of the tendency of bends to shift downstream (p. 57). For a weir there is no particular advantage in selecting a narrow place if it is also deep. In a hard and stable stream there is little restriction as to site. If, in such a channel, there is a narrow place there

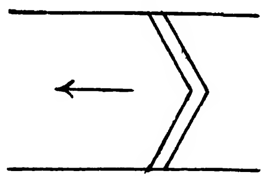


FIG. 106

may be an advantage in placing the weir there or just above it, because the water-level there is already raised and the additional raising needed is less than if the weir were placed elsewhere.

An inherent defect of an ordinary weir is that it obstructs the passage of floods. Attempts have been made to partially remedy the evil by placing the weir obliquely to the stream, thus giving it a greater length. It has been shown (*Hydraulics*, Chap. IV., Art. 18) that the advantage gained by this method is illusory or at best slight. Oblique weirs are made as in Fig. 106 or in one straight line. If the weir is lengthened, not by being built obliquely but by a widening of the stream at the

site, the crest has to be raised and little is gained. Some advantage can be gained by attending to the form of the cross-section of the weir, as will be seen directly.

The only arrangement by which a weir can be made to hold up water when a stream is low and to let floods pass quite freely, is having part of the weir movable, that is consisting of gates, shutters or horizontal or vertical timbers, which can be manipulated to let floods pass or to regulate the amount of water passing. Such weirs are dealt with in Art. 4.

The tendency for silt to deposit upstream of a weir has been already mentioned (p. 54). When deposit of sand or mud is feared horizontal passages, known as "weep holes" are sometimes left in the weir at the level of the upstream

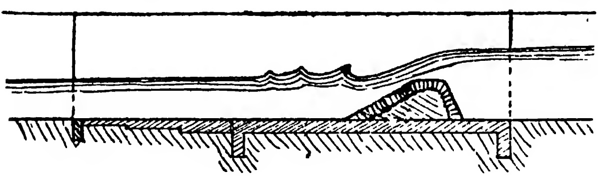
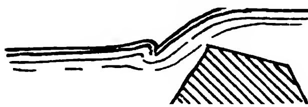


FIG 107.

bed. In the old Nile barrages iron gratings were provided. Such passages, unless they can be closed, are inadmissible in cases where the whole of the supply has to be diverted at times of low water. If a deposit of silt does take place for a long distance upstream of a weir the flood level in that reach may be somewhat raised. At the weir itself it is not likely to be much raised The crest is not raised and the velocity of approach is increased.

The upper corners of a weir should be rounded (Fig. 107). This prevents their being worn away; but the rounding of the upstream corner has another advantage. If the corner is sharp, the stream springs clear from it and the weir holds up the water higher, especially in floods. With small depths of water the difference is slight, and it vanishes when there is only a trickle of water. Thus a crest rounded on the upstream side holds up low water as well as a sharp-edged

crest but lets floods pass more freely. Reducing the top width, making the top slope upwards (Fig. 108) and giving a batter to the upstream face have similar advantages. The rounding of the crest is of more importance as the batter is less. To secure full advantage the top should be fully rounded



(Fig. 109) and the upstream face should be sloping. For similar reasons, the upstream wing walls should be splayed, or even curved so as to be tangential to the side walls, and not built normally to the stream These advantages are sometimes lost sight of, even when they are important as in waste weirs for reservoirs

Downstream of a weir there is always much eddying and disturbance of the water Unless the bed and sides of the channel are of rock, a weir has (Fig 107) side walls and rests on a strong floor or "apron" These need not extend far upstream, but must extend some way downstream. Downstream of the floor there is paving or pitching of the bed and pitching of the sides The kind of weir shown in Fig 107 is extremely common

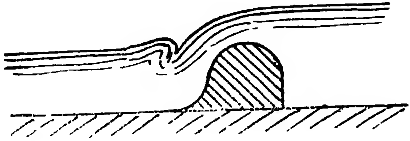


Fig. 109

The body of a weir is commonly of rubble masonry and the facework of dressed stone In large weirs the stones are sometimes dowelled together. Where, as in many parts of India, stone is expensive, brick is used for small weirs, the crest and faces being brick on edge.

Frequently a "salmon ladder" has to be provided at a weir. It consists of a series of steps or a zigzag arrangement so that the velocity of the water is not too great for the fish to ascend.

Three types of weir are shown in Figs. 110, 111 and 112.

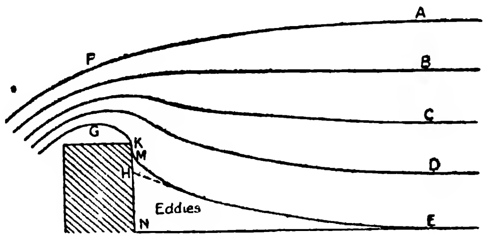


FIG 110

There are of course intermediate types and innumerable varieties in details. The weirs shown in Figs. 109 and 107 differ little from the first and second types respectively. The first two types are used chiefly when the channel is of hard impermeable material — such as shale or clay or well-compacted gravel, shingle or boulders — the third type when it is porous, but there are exceptions to these rules.

A weir is exposed to several dangers which have to be guarded against in its design and construction. They depend on the nature of the channel and on the type of weir. Danger

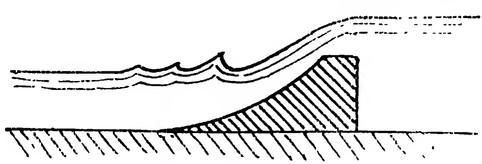


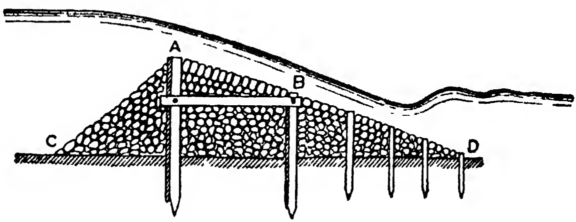
FIG 111

from downstream scour is nearly always present. It is greater the softer the channel.

In the case of the first type of weir there is the heavy shock of the water falling on the floor, and there is much commotion immediately downstream of where it falls. The

floor in these places must be exceptionally heavy and strong unless there is a cistern, see below. Swirling and waves continue further downstream. In the second type there is very rapid flow over the curved portion of the weir. The commotion though less severe than in the first type, extends further. In the third type there is a great area of weir surface exposed to a rapid current. There is disturbance over the lower part of the area..

In the case of a weir on sand or other porous material — gravel, shingle and boulders may be so — water percolates under the structure. There is danger from "uplift", that is the upward pressure of the water beneath the floor, and from "piping" or the formation of streams under the floor. Uplift



may crack a floor and so render it liable to be blown up if the uplift continues, or to fall in if piping occurs

In a weir of the first type the weir wall — or "drop wall" — has to resist being overturned or sheared by the pressure of the water. The resultant stress on it must pass through the middle third of the thickness This matter — including the pressure due to ice and pressures on foundations — are dealt with in Chap. IX.

Regarding the dynamic pressure of the water, it is shown in *Hydraulics* that the velocities at the cross-section AE (Fig. 110) are those due to the ordinary flow of the stream. Let them be 10, 8 and 6 feet per second at A, C and E respectively. The space ENK is occupied by eddies. The stream may be supposed to be divided into layers as shown by the curved lines. At F and G the velocities have become say 16 and 20

feet per second respectively. The increase in velocity is due to the fall A F, in the water surface. At intermediate points the velocities can be estimated from the diagram. At N, H the pressures are very little more than the hydrostatic pressures. The horizontal dynamic pressure on the weir may be taken to be that on HK and to be due to the change in direction, from EH to HK, of the stream passing DE. This dynamic-pressure is never very great. It is greatest in a high flood when the water-level is much higher than A and all the velocities are increased by reason of increased surface fall from A to the weir. The direction of the lower stream lines is not much affected.

If the access of air to the space downstream of the weir is prevented a partial vacuum is formed. The overturning and shearing effect due to this may be appreciable and can be calculated, a complete vacuum being assumed, but the side walls should be so arranged as to admit the air. Failures of weirs generally occur in floods or at times of considerable flow but the causes may have been long at work. The destruction of a weir may be due to overturning or shearing of the drop wall, uplift, piping, or downstream scour or any or all of these.

The dangers to a weir, and consequently the difficulties in designing it, are by far the greatest in cases where the channel is porous and particularly where it is of sand. These cases will be considered in the next Article The rules given for the length and weight of floor and length of pitching will assist in deciding similar points in channels which are not porous, though in the latter the dimensions are largely matters of experience and judgment

In a weir like that shown in Fig. 112 there is a standing wave — more or less complete — near the foot of the slope in low stages of the river and higher up in higher stages. At still higher water the disturbance due to the weir may be hardly perceptible. The weir is designed so that the standing wave comes well within the weir area. One reason for adopting the long downstream slope — instead of a steep drop and a flat apron — is that far more of the masonry can be

laid in the dry. The type is adopted where plenty of boulders can be obtained.

The type of weir shown in Fig. 107 may be varied by altering the slopes of one or both faces. Flattening the slopes may be combined with a decrease in the width of the crest. In a small stream or in an irrigation distributing channel, a weir may be a simple brick wall with both faces vertical and corners rounded. Of the other weirs mentioned below some are cheap and cannot be expected to stand heavy floods.

The "beaver dam" shown in Fig. 113 is made while the

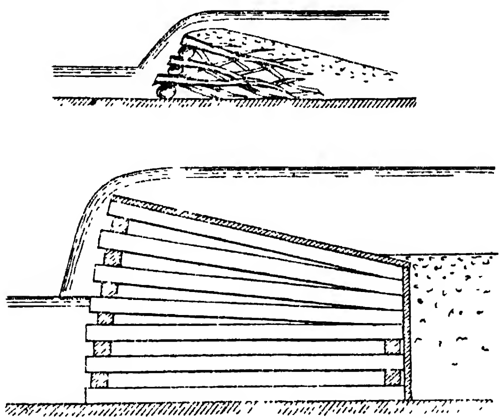


Fig. 113 and 114

stream is low. The lowest layer is of logs which are kept in position by large stones. The next layer is of trees with their branches and is spiked to the first layer. Stones and gravel are added to each layer. Such dams have been made to heights of 15 feet.

In a crib-work weir (Fig. 114) the timbers which are parallel to the stream are notched and bolted to the others. The crib is made on the bank and is floated out and filled with stones till it sinks. The top and the upstream side are planked over. Large weirs can be made by a number of such

cribs. In another kind of crib-work weir (Fig. 115), generally used on a bed of rock or gravel, a number of frames, as shown, are placed in the stream and connected by stout planking. There are other varieties. If constantly under water the timber does not rot. Somewhat similar weirs have been made of steel.

Weirs are also made of sheet piling filled in with rubble, and the top may be protected by sheet iron. A weir made on

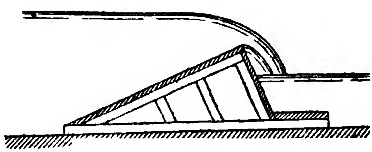


Fig. 115

the Mersey in connection with the Manchester Ship Canal works was so made. There were three rows of piles and the filling in the back part was of clay.

Art. 2. Weirs on Porous Soil. Let AB (Fig 116) be the bed of a channel, the underlying material being sand of uniform character and great depth Suppose the bed to be

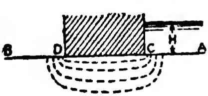


Fig. 116

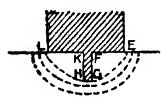


Fig. 117

covered by a thin impervious layer except for minute lengths at C and D close to the impervious obstruction CD. Percolating water will follow the uppermost dotted line. The line is vertical at its extremities and then curves sharply. The rest of it probably sags slightly — instead of being straight — owing to the effect of gravity. If the pervious lengths are slightly increased an additional percolation stream will be set up, as shewn by the second line. In order that this may be parallel to the first line the radii of the curves near the ends must be greater. The same applies to each succeeding line.

The lines probably tend to be semi-ellipses [1]. If the impervious covering is wholly removed, the lines multiply indefinitely but the velocity of flow along each line is of course less as its length is greater It is given by the formula

$$V = K \frac{L}{H}$$

where L is the length of the path traversed and K is a constant which varies with the class of sand [2]

Experiments made many years ago by Clibborn [3] show that in practice, percolating water tends to follow the surface of any hard body. Piping — perhaps slight and temporary — may occur at any place. If it occurs close to a hard body, the sand is prevented from closing in on that side. If it occurred along any line immediately beneath the body CD, it would very likely be permanent. The percolation there would follow the straight line CD If the piping increased, the lower lines of flow would be more or less suppressed. Thus in a wide stream the flow might in some parts of the width be along CD and at others wholly in curved paths.

In the second case shewn (Fig 117) the paths of the percolating streams are altered by the curtain wall. Here again the uppermost percolating stream may follow the curved line EGHL or the line EFGH. If the sand along KL separates at all from the body above it, flow may be established along HKL.

The above considerations have an important bearing on the case of weirs in porous soil. In Fig 118 let the downstream channel be dry — or carrying only the slight discharge due to water percolating under the apron and coming out at C — and the upstream water level with the crest of the weir. This condition gives the maximum water pressure [4]. The flow is as in a pipe KBNC If the pipe is uniform the hydraulic gradient is the straight line AC. It is necessary

[1] This conclusion appears to have been arrived at by mathematical analysis

[2] Notes on Sand are given at the end of this Chapter

[3] *Indian Railway Board Technical Paper*, No 97 Experiments on the Passage of Water through Sand.

[4] If a deep scour hole occurs downstream of a weir and if the water-level in the hole falls below the bed level the hydraulic gradient is increased

to consider the probable effects of the flow on the sand and also the upward pressure on the floor. These matters have been discussed by Bligh [1] and Griffiths [2].

Regarding the upward pressure on the floor due to the hydrostatic pressure, the weight of any portion of the floor PM, should be able to balance the pressure due to a head of water RM. This, supposing the masonry to be twice as heavy as water, would give a thickness of floor PM, equal to half RM, that is equal to RP. According to Bligh, the theoretical thickness ought, for safety, to be increased by one third. This rule agrees fairly with practice. If the tail water covers the floor to a depth d, this head is of course deducted from the head causing upward pressure on the floor.

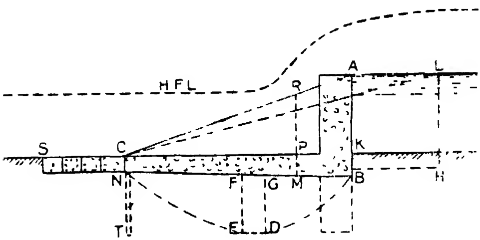


FIG. 116

Supposing the floor to be heavy enough — as it nearly always is — to safely resist upward pressure, the chief danger to the weir arises from piping along KBNC. If this becomes excessive the weir may be destroyed. The hydraulic gradient must be sufficiently flat to prevent this.

The following are given by Bligh as safe hydraulic gradients (s):

Fine silt and sand as in the Nile or Mississippi (60 per cent. passing through a 100-mesh sieve) .	1 in 18
Fine micaceous sand as in Colorado and Himalayan rivers (80 per cent. passing through a 75-mesh sieve)	1 in 15

[1] *Engineering News*, 29th Dec. 1910. Also *Dams and Weirs*, Arts. 113—136.
[2] *Proc. Inst. C. E.*, Vol. CXCVII. pp. 221—231.

Coarse sand (Central and S. India)	1 in 12
Gravel	1 in 9
Shingle	1 in 8
Boulders	1 in 5

The best method of flattening the hydraulic gradient and at the same time reducing the upward pressure on the floor is to add an impervious upstream apron BH. The gradient becomes LC Since this apron does not have to sustain the shock of falling water it can be of puddle protected on the top by pitching This may be of concrete blocks or of dry rubble The former is the better protection both generally and against cray fish and the like.

Bligh states as an empirical rule that in order to provide efficiently against downstream scour the length of floor should be $3\sqrt{\frac{H}{s}}$, where H is the maximum head AK. This length is less than that necessary to give a hydraulic gradient of the requisite flatness according to the rules just quoted The balance is available for the upstream apron. Such an apron is now usually made in the case of weirs on sand.

In the paper above mentioned by Griffiths, it is pointed out that the stability of the sand depends on the specific gravity of the grains, on their size and degree of consolidation and on the pressure between the grains, and that in the bed of a stream the consolidation is generally good and the weight of a structure is not likely to improve it much.

If a number of small parallel streams are formed under the floor, and if one of them near the upstream end of the weir, say in BG, becomes enlarged, the hydraulic gradient line in it is flattened, raised at G and the pressure increased. It is pointed out in the paper that this tends to cause flow from the enlarged stream to the others and so to restore the original conditions and do away with the piping, but that if the enlargement occurs along the downstream end FN, the pressure at F is reduced, the small streams tend to flow towards the enlarged one and the piping to be increased and become

established. In some experiments made on models[1] —very few such experiments have been made — the line of hydraulic gradient near A tended to be steep while the rest of it was flat. This in some degree points to piping below the downstream part of the weir.

Consider now the effect of a curtain wall such as GDEF. The hydraulic gradient is flattened, The water follows the path indicated by the dotted line KBDENC or else it flows down GD and up EF. Bligh, in his investigations as to gradients, assumes the existence of the latter path His figures are generally accepted In any case it is clear that the curtain wall adds to the stability of the weir. It forces the percolating streams to traverse sand which is subject to increased pressure and is better consolidated. It seems reasonable to accept Bligh's gradient figures but, for additional safety, to calculate the gradients on the supposition that the water flows down the curtain wall but not up it

It is pointed out by Griffiths that if a curtain wall is made in the position GDEF in Fig. 118 the hydraulic gradient is raised upstream of it and the upward pressure on the floor is increased The best position for it is exactly under the drop wall of the weir as shown by dotted lines The pressure on the floor downstream of the wall is then decreased. If a curtain wall NT is made near the downstream end of the floor it is as if the point C were shifted downstream The upward pressure on the floor upstream of NT is increased This is said to be an objection to such a wall but it obviously adds to the stability of the weir. The percolation line is shifted from EN to ETN The floor can be made of the requisite thickness. In any case a shallow curtain wall is necessary at NT in case the bed pitching is shifted and a hole scoured

If a curtain wall is not quite impervious to water — for instance if it is made of sheet piles which do not fit closely together — its effect is reduced. It may be largely reduced. The whole of the water percolating under a weir moves so slowly that it can pass through small crevices with little

[1] *Trans. Am. Soc. C. E.*, Vol 53 p 000

loss of head. Curtain walls should be impervious. A wall such as NT is not necessarily of the same depth as the upstream one. If a third wall, intermediate between the other two, is made its effect on the hydraulic gradient is not likely to be considerable unless the length BN is great.

In wet soil, deep curtain walls frequently consist of rows of rectangular wells sunk very close together, the space between the walls being filled by piles. Piling can be of interlocking steel sheet piles or of reinforced concrete.

Griffiths considers that small walls or ridges projecting from underneath the floor and running parallel to the stream, will tend to discourage the concentration of percolating streams. Others consider that several shallow curtain walls are better than a few deep walls. The best system of all is to construct a few deep curtain walls. When the expense of this is prohibitive the other methods can be adopted.

The wing walls of a weir should be so designed that water percolating round the flanks of the weir will not find a steeper hydraulic gradient than that suited to the soil. The soil may not be the same as that of the bed. Griffiths states that the line of hydraulic gradient under a weir should be below spring water level. If the sand became dry and water was suddenly admitted to the channel piping might occur. Also that in the case of quicksands — which are fine light sands — existing near a work, the weight of the weir should suffice to prevent their existing below the work but that otherwise the weight can be increased by means of crates filled with stones laid over the whole area, the impervious apron being then added.

It has been seen (pp. 140—141) that in time an earthern bank generally becomes practically water-tight especially if the water contains silt. But any considerable leakage of water is observed and, if necessary, remedied. With a weir this cannot be done. A weir has no doubt the same general tendency to become water-tight but while this process is going on there may be piping at some particular place.

The floor of a weir is, as has been seen, impermeable. The downstream pitching is permeable. The length CS to

which pitching, if of "rip-rap" type, should extend is given by Bligh as $\frac{10}{s}\sqrt{\frac{H}{10}}\sqrt{\frac{q}{75}}$, where q is the maximum discharge in cubic feet per second passing over a 1-foot length of the weir, and H is the head AK.

In a few cases a portion of the floor next the pitching has been made slightly permeable, being known as a "filter" and consisting of a layer of gravel over which are stone spalls and over them rip-rap. The idea is that the water percolating upwards will carry up sand and gradually render the filter water-tight. The cost is of course less than that of a solid floor.

In the case of weirs on permeable gravel, shingle or boulders the dangers are of course far less than in weirs on sand. The principles of design and construction are the same In many cases a gradient which is quite safe may be unsuitable because of excessive leakage.

Porous Weirs. — A weir may be of loose stone with a few cross walls of stone (Fig 125) or of piles (Fig. 112) or of rows of wells. The walls determine the hydraulic gradient. Referring to Fig. 112, if the upstream water is level with A, that between A and B is level with B and so on. The hydraulic gradient is ABD. When water flows over the weir, sand and silt are deposited among the boulders The whole weir becomes to a great extent impervious and it can be made without the masonry walls. Figure 112 shows a type which is used on a somewhat small scale, Fig 125 a large weir described below. The boulders settle into the sand especially in the lower part of the weir, so that the weir is not all on the top of the original bed as shown in the figures.

Art. 3. Special Types of Weir. *Falls and Rapids.* — When the reach of channel downstream of a weir has a bed-level much lower than that of the upstream reach — this is commonly the case in irrigation canals — the work is known as a "fall" or "rapid". At a fall the water generally drops vertically (Fig 119) and a cistern is provided. The falling water strikes that in the cistern and the shock on the floor is greatly reduced. An empirical rule for the depth of the cistern, measured from the bed of the downstream

reach, is $K = H + \sqrt[3]{H}\sqrt{D}$, where H is the depth of the crest of the fall below the upstream water-level, and D is the difference between the upstream and downstream water-levels. It is now usual to make the downstream side of the cistern sloping (Fig. 132) At some old falls on Indian canals the water, as it begins to fall into the cistern, is made to pass through a grating which projects upward, but inclined

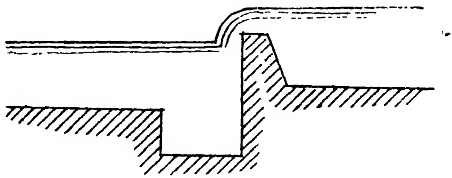


Fig. 119

downstream, from the crest of the weir at the downstream corner. This splits up the water and reduces the shock, but rubbish is liable to collect.

In the usual modern type of canal fall in India the weir has no raised crest, and the water is held up by lateral contraction of the waterway just above the fall. The "notch" through which the water passes is trapezoidal (Fig. 120) being wide at the water-level and narrow at the bed-level.

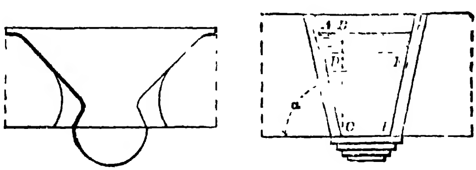


Fig. 120

In a small channel there is only one opening, but in a large canal there are several side by side, so that the water falls in several distinct streams. The curved lip shown in the plan is added to make the water spread out and cause less shock to the floor. There is also a cistern as in Fig. 132.

On Indian canals a notch fall and bridge are generally combined. The bridge is placed below the fall so as to avoid

a high roadway. The dimensions of the openings are calculated so that however the supply in the canal may vary, there is never any heading up or drawing down. The detailed method of calculation for finding CF and the ratio of AB to BC is given in *Hydraulics*, Chap. IV.

When it is desired to limit the volume of water entering a channel from a river, the channel is sometimes contracted from both sides (Fig. 121) so as to form a flume. This is really a form of submerged notch. The effect of such a work must not be exaggerated. In the case sketched the length of the narrow part of the flume was 200 feet. With 9 feet of water upstream, a surface fall of 4 inches in the flume would give a slope of 1 in 600 and a velocity of about 6 5 feet per second. An additional fall of 8 inches at the upstream end would be

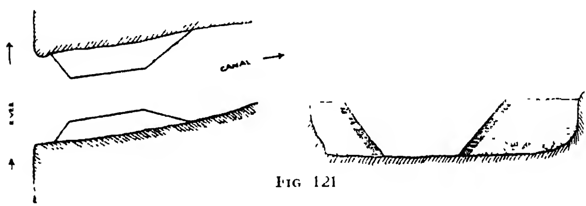


FIG 121

required to impress this velocity on the water. The flume would reduce the depth in the canal from 9 to 8 feet

A "rapid" has a long downstream slope, which is expensive to construct and may be difficult to keep in repair, especially as the canals can only be closed for short periods. Rapids exist in large numbers on the Upper Bari Doab Canal in India, the face-work consisting in most cases of undressed boulders about 1.25 feet long, placed on end, with the interstices filled up by spawls and concrete These stand the wear well. Rapids have again been used on the more modern canals in places where boulders are obtainable, and where deep foundations would have given trouble in unwatering The upstream face of a rapid is vertical, or has a steep slope. The depth of water on the crest is 3 feet and upwards. In Burma it ranges up to 11 feet A standing wave occurs as already stated

The slopes of rapids are usually 1 in 10 to 1 in 15. Before the boulder pitching is added, the whole of the slope of the rapid is usually divided, by low masonry walls running both transversely and longitudinally, into rectangles with sides of perhaps 25 feet, so that any damage to the pitching is likely to be confined to one compartment

On American distributaries steep rapids are sometimes used instead of falls At the foot of the rapid the channel is widened out and lined with concrete and forms a stilling basin.

Weirs in Great Rivers. — Weirs of very great lengths have been constructed across rivers at the headworks of irrigation canals Along the more modern weirs there are generally piers, at intervals of 300 to 500 feet, rising well above flood level and carrying cableways for traffic Along the crest there are nearly always hinged shutters (Art 4) which are laid flat during the flood season.

Most of the weirs are on sand In such cases the cost of maintenance is always very considerable. Sometimes danger arises owing to the setting up of currents parallel to the crest of the weir Some of the weirs are provided with groynes to prevent such damage

Six Indian weirs on sand are described below The hydraulic gradients are calculated by Bligh's method. The Narora weir is the only one with a drop wall The others are of the type with a long slope — 1 in 15 — from the crest to the downstream end In several cases the river bed upstream of the weir is higher than downstream because of the silting which has taken place above the weir. But the bed-level in different parts of the width of the river of course varies. Two of the weirs failed and afforded valuable evidence, assisting Bligh in his researches as to suitable gradients. In Figs. 122 to 124 the masonry or concrete work is shown hatched. The details vary Sometimes the whole thickness is concrete or boulder masonry. The face work may be of hammer-dressed stone The upstream pitching is generally of dry stone, the downstream of flat concrete blocks 2 feet thick. If the blocks are of masonry they are 2.5 ft thick.

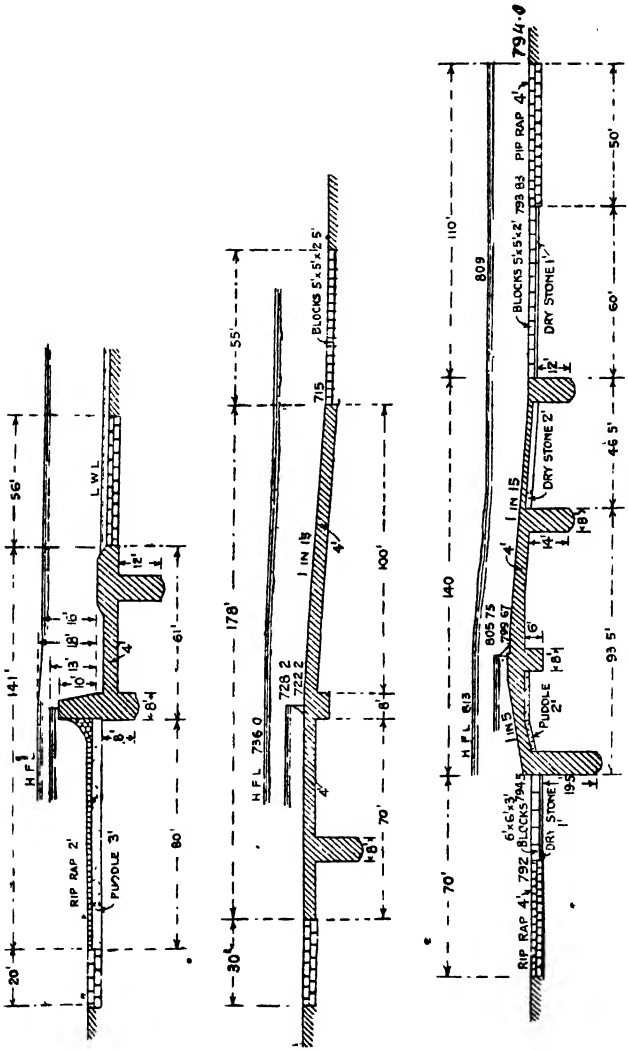


FIG. 122, 123 and 124

In the case of the Narora weir across the Ganges at the head of the Lower Ganges Canal (Fig. 122) the hydraulic gradient was 1 in 11.8 the water held up being 13 feet. The shutters did not from part of the original design but were added later. The weir stood for some 20 years. By that time the sand below the floor had been washed out and the floor was held up by the water pressure. In a flood, cross currents appear to have been set up and that part of the upstream apron nearest to the drop wall was destroyed The hydraulic gradient thus became 1 in 8 and the floor — in a length of 350 feet of the weir — was blown up. This was not in the flood season The weir was reconstructed with the upstream apron 80 feet wide instead of 38 feet. The floor below the drop wall was made thinner than before, but with 2 feet of water over it to act as a cushion The hydraulic gradient is 1 in 15, both faces of the piling and of the walls below the floor being counted in the length.

In the Khanki weir across the river Chenab at the head of the Lower Chenab Canal (Fig. 123) there was at first no upstream apron. The gradient was 1 in 8 3. After some years a portion of the weir failed owing to piping. An upstream apron with a curtain wall of wells was added to the whole weir the gradient being reduced to 1 in 16 In some portions of the weir the apron is level as shown. In others the upstream end of the apron is 3 feet lower than the crest of the weir

The Rasul weir is in the river Jhelum at the head of the Lower Jhelum Canal. It has two deep curtain walls — one at the crest and the other half-way down the slope — and a shallower wall at the toe of the slope The weir has been described as being merely a "bar" — except when the shutters (6 feet high) are up — its crest being at about the same level as the general bed of the river at the time of construction. The hydraulic gradient is 1 in 20.3.

The latest Punjab weir is the Merala weir (Fig. 124) across the Chenab at the head of the Upper Chenab Canal. The river here is liable to very heavy and sudden floods. The upstream apron and downstream pitching are of great

extent and there is ample provision of deep curtain walls. The total width of the weir including the above is 320 feet and is greater than that of any other known weir. The hydraulic gradient is 1 in 20.6.

The Rupar weir across the Sutlej at the head of the Sirhind Canal was originally a boulder weir with two cross walls. It has since been faced with masonry and is now of the same type as the weirs just mentioned. The downstream slope is 1 in 15. Its height is 9.5 feet above the original river bed and there are 6-foot shutters. The hydraulic gradient is about 1 in 12.5.

A very old weir is that across the Godavari at the head of the delta canals. The length is about 12,000 feet. The bed of the river is coarse sand. There is a short slope and

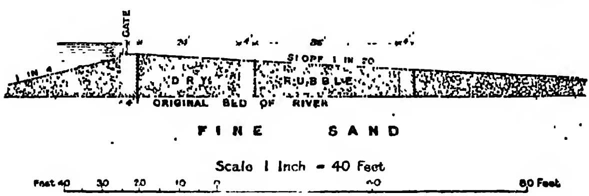


Fig 125

a steep hydraulic gradient — say 1 in 10 — and very heavy and wide downstream pitching of boulders. The maintenance has been expensive involving the expenditure of large quantities of boulders.

Another group of weirs are also on sand but they are of the boulder type. The Okla weir is the only one which has shutters.

The Okla weir (Fig. 125) across the river Jumna at the head of the Agra canal was built about fifty years ago. The river bed consists of fine sand. The depth of water over the crest in floods is 6 to 10 feet. The hydraulic gradient is 1 in 15.8.

The Dehri weir across the Son river at the head of the Son Canals, has a length of no less than 12,500 feet. The slope is 1 in 12. It is similar to the Okla weir but has two

curtain walls going down to 7.5 feet below the river bed. The sand is coarse. The high flood level is 9.2 feet above the crest. Bligh compares the design with that of the Okla weir and considers that an enormous saving in cost could have been effected, by omitting those parts of the walls which are below the river bed and that three more walls could have been added and the upstream slope made impermeable and thickened at the apex. The hydraulic gradient is 1 in 12.

The Laguna weir over the Colorado river in the United States, at the head of the Yuma canal, has a slope of 1 in 12. There are only 3 walls. They do not go down below the bed but the upstream wall has a line of piles under it. The depth of water on the crest in floods is only 5 feet. The hydraulic gradient is 1 in 13.6.

The weir across the river Kistna in Madras is also of boulders The slope is about 1 in 12

The weirs across the Rosetta and Damietta branches of the Nile are also of boulders but very heavily built.

A few of the great weirs -- mentioned below — are on hard beds.

The weirs across the Ravi at Madhopur at the head of the Upper Bari Doab Canal, and across the Jumna at Tajawalla at the heads of the Eastern and Western Jumna Canals, are on hard beds of boulders and shingle and — though they are liable to be damaged in floods — give much less trouble than weirs on sand

The cross-sections are similar to that of Fig 107 (p. 236) with the downstream slope flattened to 1 in 10. The bodies of the weirs are of ordinary boulder masonry, the face work of dressed boulder masonry

The most recent headworks weir in India, that at Bhimgoda (Hardwar) across the Ganges at the head of the Ganges Canal, is of a new type (Fig. 126). It is of boulder masonry faced with hammer-dressed boulder masonry and at the top corners with granite.

Art. 4. Weirs with Movable Parts. *General Description.* — When a weir has movable parts the latter may be of any of the types described below but are usually gates

which slide vertically in grooves. A weir may consist wholly of movable parts and is then — if in a river — often termed a barrage or dam or a sluice or sluices. When in an irrigation canal it is called a regulator and is usually placed at a bifurcation. Any such work when fully opened, is no more than a bridge and offers little obstruction to the stream.

In all kinds of sluice openings or regulators, the principles of design as regards protection of the bed and sides, splaying and curving of walls and piers, thickness of floor and prevention of the formation of streams under the structure are the same as laid down for weirs. The floors are actually, in many cases in India, much thinner than would be given by the rules. The reason for this is that the long piers and side walls help to sustain the floor, which can act as an arch or

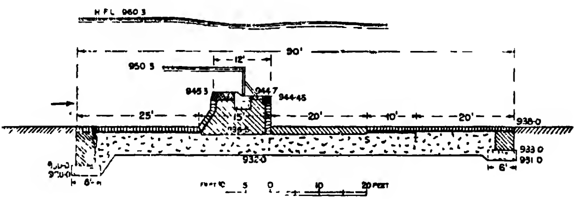


FIG. 126

as a beam — the lime used is generally excellent — and the thicknesses adopted have been found in practice to suffice.

In order that a pier may be safe from being overturned by the pressure of the water when the gates or timbers are in use, the resultant of its weight, including that of anything resting on it, and of the water pressure on it, must pass through the middle third of its length. This generally occurs when there is an arched roadway. Otherwise it must be arranged for by prolonging the base of the piers downstream and giving the downstream side a batter or steps.

Sluices with gates are, of course, used in connection with works other than weirs or regulators, as for instance in reservoirs or locks, for scouring or escapage or generally for communication between any two bodies of water. The gate may or may not be wholly submerged. If it is not wholly sub-

merged planks can be used if convenient. Needles (see below) can be used if the flow is always in one direction. Sluices with Stoney gates are used on the Manchester Ship Canal (p. 171) where the water of the river Weaver is passed across the canal, and at locks for passing the flood waters of the Irwell and Mersey down the canal. In all cases protection downstream of the opening is required. When sluices are used for regulating the escape of water from a reservoir the arrangement is often called an outlet. These are dealt with in Chap. IX.

The long weirs built across Indian rivers below the heads of irrigation canals generally extend across the greater part of the river bed. In the remaining part — generally the part nearest the canal head — there is, instead of the weir, a set of openings or "under-sluices" (Fig. 127) with piers

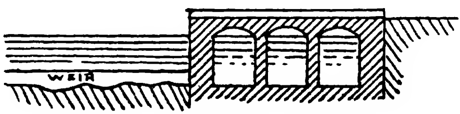


FIG 127

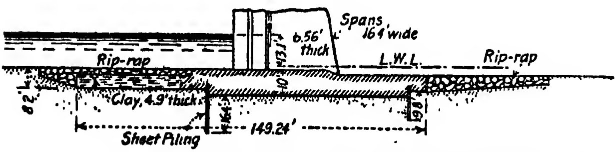
having iron grooves in which gates can slide vertically. The piers are often twenty feet apart and five feet thick. The gates are worked by one or more "travellers", which run on rails on the arched roadway. The traveller is provided with screw gearing to start a gate which sticks. When once started it is easily lifted by the ordinary gears. The gates descend by their own weight. Sometimes instead of a traveller there is a set of fixed winches. The gate in each opening is often in two halves, upper and lower, each in its own grooves, and both can be lifted clear of the floods. In intermediate stages of the river these gates have to be worked a good deal. Usually the weir has, all along its crest, a set of hinged shutters, 5 or 6 feet high, which lie flat at all seasons except that of low water in the river.

The canal head consists of smaller arched openings, provided with gates working in vertical grooves and lifted by a light traveller. If, as is usual, the canal head has a "sill"

higher than the beds of the river and the canal, such sill is a weir, but otherwise the canal head is merely a set of sluices without a weir.

The barrage of the Nile at Assiut (Fig. 128) and the old barrages of the Rosetta and Damietta branches, consist of sets of sluices without weirs. At Assiut there are piers five metres apart, and gates working in grooves like those, above described, at Indian headworks. The heading up is 10 feet and the hydraulic gradient 1 in 19. The thickness of the floor is ample. Fig. 129 shows the Zifta regulator on the Nile. The hydraulic gradient is 1 in 16.4.

In the more modern canals the spans both of the undersluice openings and of the head regulators are increased and Stoney gates (described below) are used. At the recently



FOUNDATION OF THE ZIFTA REGULATOR, RIVER NILE.

FIG 129

completed headworks of the Ganges Canal the spans of the undersluices are 50 feet and those of the regulator 20 feet.

Generally the head regulator of a canal is at right angles to the weir and the head reach of the canal runs out at right angles to the direction of the river. In irregular country and with rivers which are of moderate size and have little tendency to shift their channels, the head reach of the canal sometimes skirts the river — high ground or hills may prevent its getting away — and the head regulator is made in the same line as the weir or else the canal takes a sharp turn immediately below the regulator. With rivers of moderate size the sluice gates generally extend entirely across it. Sometimes a high weir extends entirely across it and is pierced near the canal head by a few rectangular openings.

Ganges Canal Undersluices Stoney Gates

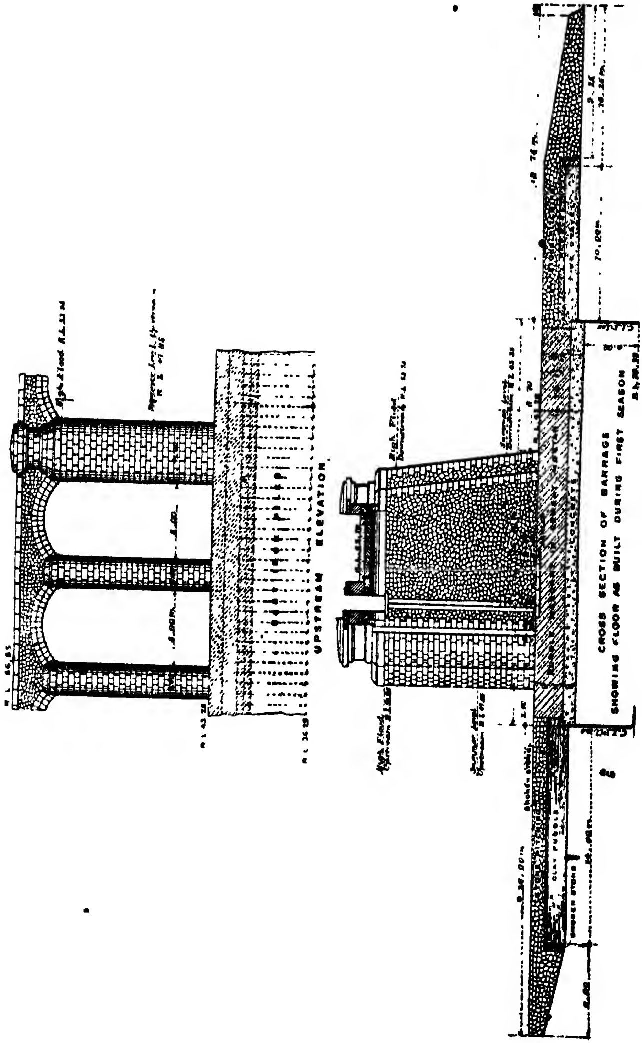

UPSTREAM ELEVATION
CROSS SECTION OF BARRAGE
SHOWING FLOOR AS BUILT DURING FIRST SEASON

FIG. 128.

At the headworks of the Upper Jhelum Canal (Fig. 130) the high flood-level is 60 feet above the sill of the regulator and the range of the water level is 40 feet. The regulator is founded on rock. It has twenty bays of 12 feet each. In each bay there are 3 Stoney gates, 6 feet 6 inches, 6 feet 9 inches and 8 feet high, operated by overhead winches. Each group of 4 bays has one winch. To it 1, 2 or 3 gates in a bay can be attached. The middle gate can be raised independently

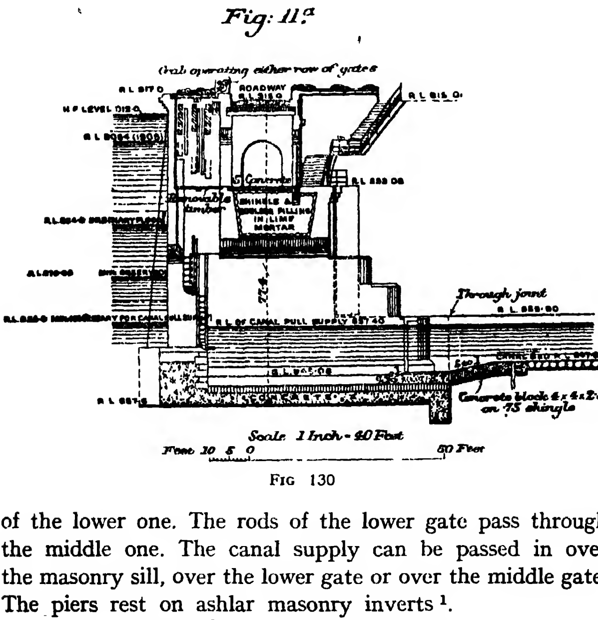
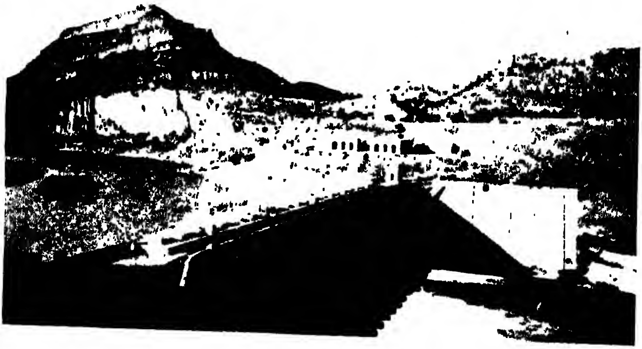


FIG 130

of the lower one. The rods of the lower gate pass through the middle one. The canal supply can be passed in over the masonry sill, over the lower gate or over the middle gate. The piers rest on ashlar masonry inverts [1].

Where a great supply of canal water had to be carried across the Ravi a syphon was at first proposed — an escape would probably have been included — but the cost would have been too great, and a level crossing (Fig. 42, p. 106) was provided. It forms the headworks — 15 bays of 20 feet — of the Lower Bari Doab Canal and the tail inlet —

[1] *Proc. Inst. C. E.*, Vol. CCI. Plate 3.



Mangla Headworks, Upper Jhelum Canal, India

Under Sluices on the river Moo, (Burma) Span of each gate, 40 ft.
.. Stoney Gates, Ransomes & Rapier, Ltd.. London and Ipswich.

Ravi Barrage

15 bays of 10 feet — of the Upper Chenab Canal. In the Ravi there is a barrage of 35 bays of 40 feet, with counterbalanced 12-foot 6-inch gates. The regulator has upper and lower gates and is designed to take in only top water. The Ravi discharges 200,000 cubic feet per second in the floods but may be nearly dry in the winter. The river barrage is of course just below the canals. The canal water can spread up the river so far as the level of the river bed permits. It would not have been economical to construct a work to prevent the spread. When no water is coming down the river the percolation of the spill water has, however, to be made good by the canal supply. This is one disadvantage of the level crossing. Another is the expense of working the barrage and regulator. The weir is of the same type as the Merala weir, the crest being 1.4 feet above the upstream bed and 6.9 feet above the downstream bed and the slope 1 in 15.

The gates are worked by the Ashford Patent Rapid Lifting Gear made by Glenfield & Kennedy, Kilmarnock. An operating shaft traverses the length of each span. At each end of the span is a lifting drum keyed to a short shaft and carrying a toothed wheel which gears with a pinion keyed to the operating shaft. At the centre of the operating shaft is the headstock carrying worm reduction gearing and also a clutch which can be easily withdrawn under load. The drums — on which the steel wire ropes coil — are fitted with double eccentric pads greater in diameter than the drum. When the gate is down it hangs from the drums, the balance box from the eccentric pads. When the clutch is withdrawn, the gate runs up. When nearly up the gate ropes mount similar eccentric pads and the gate comes to rest without shock. The gates can thus be raised with great rapidity in case of need.

The "dam" across the Ravi, at the head of the Sidhnai Canal in the Punjab, consists of sluice openings without a weir. The piers are connected by horizontal beams (Fig. 131) against which and against a sill at their lower ends, rest a number of nearly vertical timber "needles", fitting close together, which can be removed when necessary by

men standing on a footbridge. In floods the needles are all removed and laid on the high-level bridge (not shown in the drawing) the footbridge being then submerged. With needles the span between two piers can be greater than would be possible with a gate. Needles can be used up to a length of 12 or 14 feet, excluding the handle which projects above the horizontal beam. They can be of pine, about 5 to 7 inches deep in the direction of the stream and 4 inches thick.

The regulators at canal bifurcations generally have two sets of piers — one in the canal and one in the branch — with openings and gates like those at the canal heads, or else with wider openings and needles. The gates are worked by travellers or by fixed winches. For the smaller branches

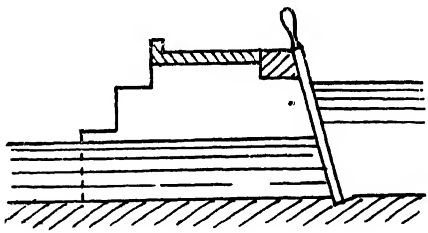


FIG 131

the gates are lifted by pinions working on racks attached to the gates, or else the gates are replaced by sets of planks or timbers lying one above another and removed by means of hooks. They are inserted by means of the hooks or by being held in position some little height above the water and dropped. They are finally closed up by ramming. Very small gates for distributaries are often worked entirely by screw gearing.

Barrages and weirs with movable parts are not, of course, confined to canal headworks. At Teddington on the Thames the oblique weir, 480 feet long, has thirty-five gates which extend over half the length of the weir. They are worked by travellers which run on a footbridge. The openings do not extend down to the river bed but are placed on the top

on a low weir. The other half of the weir is fixed. The gates are raised to let floods pass.

At Richmond on the Thames the arrangements are similar, the gates being counterbalanced to admit of easy and rapid raising. When raised they are tilted into a horizontal position so as not to obstruct the view.

A design for a regulator at a bifurcation on a recent Indian canal is shown [1] in Figs. 132. There is a cistern as for a fall.

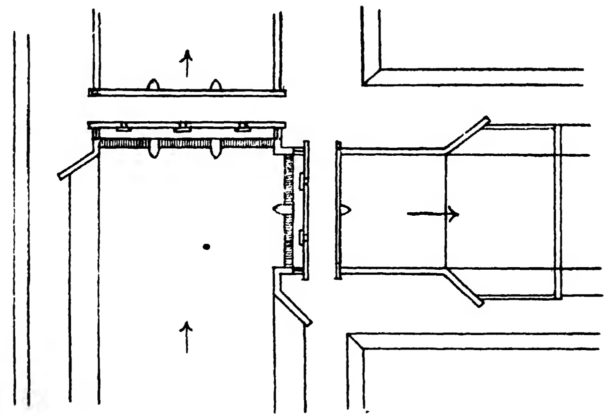


FIG 133

Another design — a needle regulator on an inundation canal — is shown in Fig 133. This has merely a level floor which is perhaps a foot lower than the canal bed (p. 283). The scale is 30 feet to an inch. The spans are 15 feet. The roadway is on arches but the regulating platform on steel beams. The needles are seen upstream of the regulators. They are worked from the platforms to which access is obtained through the gaps in the upstream parapets. The regulating platform is generally only just clear of the full supply level, and therefore lower than the roadway.

A head for a distributary or other channel of moderate size has already been described and illustrated (pp. 78 and

[1] *Proc. Inst. C. E.*, Vol. CCI. Plate 4.

134). Instead of arches steel beams can be used. The head shown has only one set of grooves for the insertion of the regulating planks. It is better to have two sets. Only one set is ordinarily used, but when the distributary has to be closed for silt clearance and all leakage stopped, both sets of grooves can be used and earth rammed in between the two sets of planks.

The head of an irrigation outlet or watercourse is shown in Fig. 134. In American canals the distributary heads or "turnouts" are sometimes of wood. The watercourse heads sometimes have sloping faces — coinciding with the inner slope of the distributary — and sloping gates.

On some of the older Indian canals there is sometimes a regulator at a place where there is no main bifurcation.

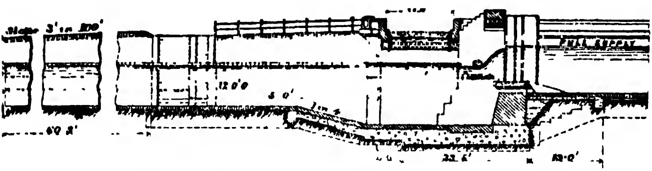
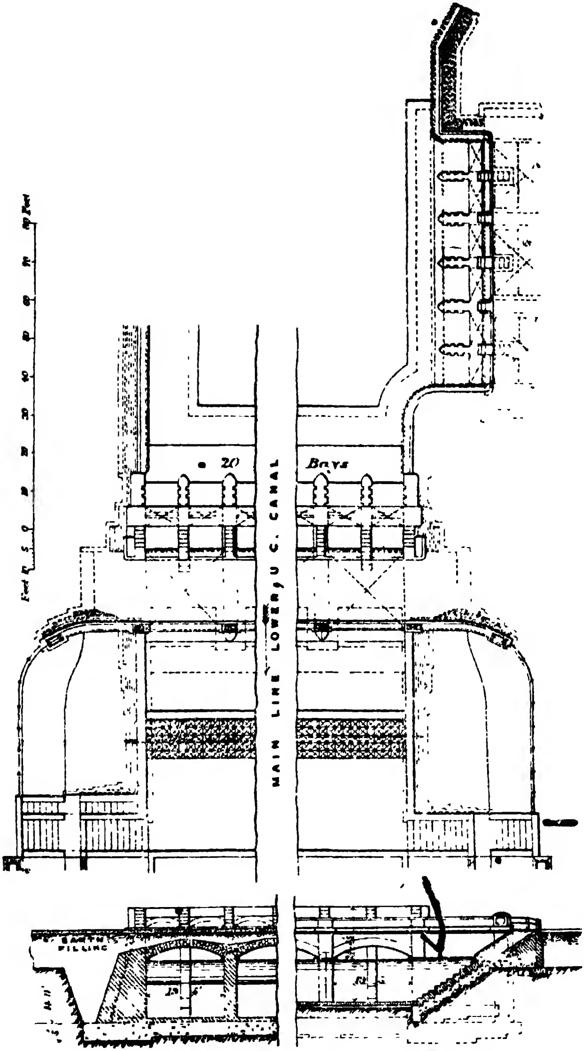


FIG. 132

There are watercourse heads upstream of it, some perhaps at a great distance. Such a structure is known as a stop-dam. In America they are also used and are called "checks".

The designs of masonry heads for escape channels (pp. 71 and 165) are similar to those of regulators. On American canals it is not unusual to provide a waste weir — on the bank of the canal nearest the river — immediately below the head regulator, the crest of the weir being at the level of the canal full supply so that if there is excess of water some will spill over. The longer the weir the better the adjustment of the supply. Such an arrangement would usually be impracticable on an Indian canal. The full supply level alters if there is silt in the canal head. Also in floods the river water, even below the weir in the river, might be higher than the canal water.

Some 30 years ago the hydraulic gradient was not con-



PART SECTION. PART ELEVATION.

FIG 132

sidered with reference to masonry works. In the case of a regulator constructed near the head of an inundation canal in the Punjab, the bed was of pure sand. The floor consisted of a 3-foot slab of concrete with its top 3 feet below the bed. The length of the floor was about 25 feet. There was also a curtain wall of sheet piling 8 feet deep. There were no rules limiting the amount of heading up at the regulator. During a flood the difference between the upstream and downstream water-levels might be 5 or 6 feet. The hydraulic gradient would then be about 1 in 9 or 1 in 8. The regulator failed during the first flood which occurred after its construction.

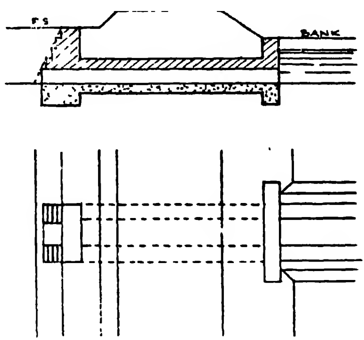


FIG 134.

Some Details. — When a canal is first made the head-works may be of a temporary nature the weirs being of one or other of the temporary kinds above mentioned and the regulating head of crates filled with stone.

When the gates are light, as in some cases of head regulators, they are when raised merely hung from short projecting brackets. From these they can be pushed off by means of a lever and they then drop into the water and probably go down on to their sills. If not they can be helped down by ramming. The light traveller which lifts the gates can be made to overhang the roadway. When the gates are heavy there is an arched rib, as in Fig. 128, on the upstream side.

traveller is carried partly by this rib and thus has the gate right underneath it. Gates can be worked electrically when this is convenient. At the Weaver sluices on the Manchester Ship Canal the traveller carries a steam engine which works the gates.

In Stoney's sluice gates, a set of rollers is interposed between the gate and the groove. The rollers are suspended from a chain, one end of which is attached to the top of the gate and the other end to the groove. The rollers move up or down at half the rate of the gate, and some of them are always in the proper position for taking the pressure. Escape of water between the gate and the groove is prevented by a rod which is suspended on the upstream side of the gate close to its end, and is pressed by the water against the pier. The spans range up to 80 feet. The gates are usually counterbalanced. To prevent the rollers chattering and thus becoming worn a shield plate can be fitted to the groove.

In the case of a needle regulator or barrage the needles can be provided on their downstream sides with eye-bolts just above the level of the beam against which their upper ends rest. They can then be attached by chains or cords to the beam or to the next pier, and cannot be lost when released. They can be released by a lever which can be inserted under the eye-bolt. By pushing the head of a needle forward and inserting a piece of wood under it, a little water can be let through. In this way, or by removing needles here and there, the discharge can be adjusted with exactness.

At a needle weir in an Indian canal all the needles in one opening are reported to have broken simultaneously. A possible explanation is that one needle broke and that the velocity thus set up in the approaching stream caused the others to break. On another occasion when a canal was dry all the needles were blown down.

Sometimes the beam or bar against which the upper ends of the needles rest is itself movable. At Ravenna, in Italy, the bar between any two piers has a vertical pivot at one pier and can swing horizontally. Its other end is held up by a prolongation of the next bar, near to its pivot. If

the end bar of the weir is released, each bar in turn is released automatically.

Frame weirs [1], used chiefly on rivers in France but also in Belgium and Germany, are a modification of the needle and plank arrangements above described. For the masonry piers there are substituted iron frames or trestles, which are hinged at the floor-level so that, when the timbers have been removed, the frame can be turned over sideways and lie flat on the floor, thus leaving the waterway absolutely clear from side to side of the stream. The footbridge which rests on the frames is removed piece by piece. The frames are raised again by means of chains attached to them. In order that the frames may not be too heavy they are spaced 3 to 4 feet apart, or very much nearer than when masonry piers are used. Horizontal planks can thus be used of shorter lengths than the needles, and they can be made up into greater widths so that the leakage is less.

A further modification consists in placing the bridge platform above flood-level, and in hinging the frames to it instead of to the floor. The frame turns about a horizontal axis parallel to the length of the weir. A weir of this kind can be used for greater depths of water than the ordinary frame weir.

In some cases the horizontal planks are connected together by hinges so that they form a "curtain". The curtain is raised by rolling it up by means of a traveller. It admits of rapid and accurate adjustment of the water-level, but there is considerable scouring action below a curtain when it is somewhat raised.

On the Trolhättan canal in Norway, instead of frames there are vertical H-beams hinged at the top and with the lower ends resting against a sill. Panels of buckled plate are inserted and are provided with rollers and staunching rods [2].

In the weir on the Thames near Eynsham there are needles which, instead of touching one another, are spaced about

[1] *Proc. Inst. C. E.*, Vols LX and LXXXV.
[2] *Engineering News Record* Vol. 83, p. 116

1½ feet apart. Their lower ends carry shutters which touch one another. The first set reaches down to the floor and the lowest part of the stream in thus closed. The next set are shorter, the shutters going down till they rest on those below and their edges resting against the first set of needles. A third and still shorter set can be added.

At falls, regulation is sometimes effected by cylindrical gates which are balanced. They are used on some American distributaries. The Yuma canal in America is carried under the Colorado River by a 14-foot tunnel. The intake gate is a steel cylinder 21 feet in diameter and 8 feet high. When down on its sill its top is 1 foot above the normal water level of the canal [1].

Sometimes a vertical gate is used which swings on a central vertical pivot so that it can be turned with its edge to the stream

Hinged Shutters. — In Thenard's system, first used in France, a shutter (Fig. 135) is hinged at its lower edge and is held up by a strut. When the lower end of the strut is pushed aside it slides downstream and the shutter falls flat. To enable the shutter to be raised again an upstream shutter, which ordinarily lies flat and is held down by a bolt, is released, and it is then raised by the current to the extent permitted by a chain attached to it. The downstream shutter is then raised. Thénard's system was not much used in France because the river had to fall to a level somewhat too low for navigation before the shutters could be raised. The sudden jerk on the chain of the upstream shutter is also liable to do damage. The system has been adopted on some of the weirs which cross Indian rivers. To prevent damage by shock, a hydraulic brake was designed by Fouracres. It consists of a piston which travels along a cylinder and drives water out through small holes.

In the Chanoine system of falling shutters (Fig. 136), used first in France, the shutter is hinged at a point rather higher than the centre of pressure. The hinge is supported by a vertical trestle, which is hinged at its lower end and

[1] *Trans. Am Soc. C. E.*, Vol. 39, pp 389—418

is supported by a strut which slides in a groove and rests against a stop. When the water rises to a certain height — dependant to some extent on the downstream water-level — above the top of the shutter, it is turned by the force of the water into a horizontal position. The struts can then be pushed sideways out of the stops by means of a "tripping bar," which lies along the floor parallel to the line of shutters and is worked from the bank. The struts, trestles, and shutters then fall flat. To close the weir the shutters are first raised into the horizontal position which they occupied before falling, by means of a hook worked from a boat or by chains attached to a foot-bridge running across the river upstream of the weir. They can then be easily closed by a boat-hook. The water closes them of itself if it falls low enough.

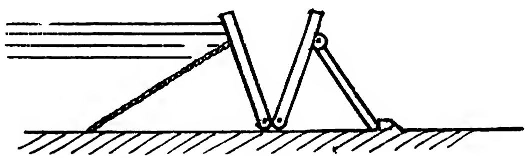
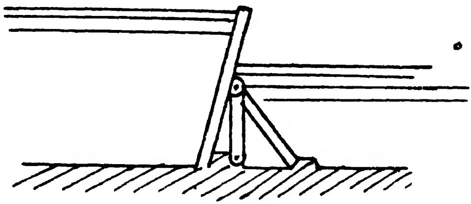


FIG 135

When the shutters fall, a great rush of water occurs. To obviate this a valve is made in the upper half of the shutter. It consists of a miniature shutter on the same principle as the main shutter. The pivot of the main shutter is made at such a height that the shutter will not turn over when only a small depth of water flows over it. Instead of this the valve comes into operation. The valve also facilitates the raising of the shutter. Again, instead of the tripping bar, which would sometimes have to be of great length or be liable to damage owing to stones jamming in its teeth, the shutter can be released by pulling the strut upstream so that it falls into a second groove, down which it slides. When a tripping bar is used, its teeth can be so arranged that the shutters are released a few at a time, first singly, then in twos and threes. Sometimes there are gaps of a few inches between one shutter and the next, and the gaps can be closed by needles if necessary.

Chanoine shutters — or "wickets" — can be very rapidly lowered, and they are used in France and in the United States in places where sudden floods occur. They are also used for navigation "passes" where most of the heavy traffic is downstream and where it is too heavy to be dealt with in a lock. A foot-bridge across the stream or across the navigation pass is always an assistance, but sometimes it



cannot be used when there is much floating rubbish or ice. With a foot-bridge the cost is greater than that of a needle weir [1].

In the Bear Trap weir (Fig. 137) the upstream shutter rests against the downstream one Both are raised by admitting water from the upper reach, by means of a culvert,

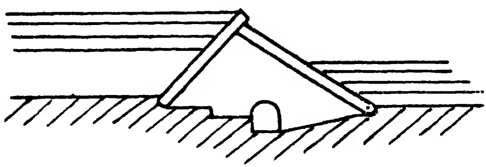


Fig 137

through an opening in the side wall, and they are made to fall by placing this opening in communication with the downstream instead of the upstream reach. This kind of shutter is only suitable for passes of moderate width — say 100 feet — and it is rather expensive on account of the culverts [2].

Shutters with fixed supports are used on the Irwell and Mersey. A fixed frame is built across the stream (Fig. 138)

[1] *Rivers and Canals*, Harcourt. *Trans. Am. Soc. C. E.*, Vol 48, p. 3. The Paper gives details as to working and describes a modification called the Bebout wicket.

[2] *Proc. Inst. C. E.*, Vol LX. *Trans. Am. Soc. C E.*, Vol 48, p. 3.

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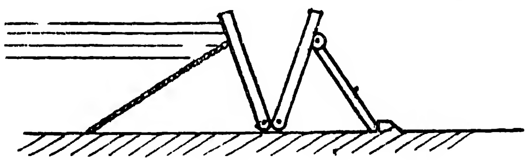

FIG. 135

When the shutters fall, a great rush of water occurs. To obviate this a valve is made in the upper half of the shutter. It consists of a miniature shutter on the same principle as the main shutter. The pivot of the main shutter is made at such a height that the shutter will not turn over when only a small depth of water flows over it. Instead of this the valve comes into operation. The valve also facilitates the raising of the shutter. Again, instead of the tripping bar, which would sometimes have to be of great length or be liable to damage owing to stones jamming in its teeth, the shutter can be released by pulling the strut upstream so that it falls into a second groove, down which it slides. When a tripping bar is used, its teeth can be so arranged that the shutters are released a few at a time, first singly, then in twos and threes. Sometimes there are gaps of a few inches between one shutter and the next, and the gaps can be closed by needles if necessary.

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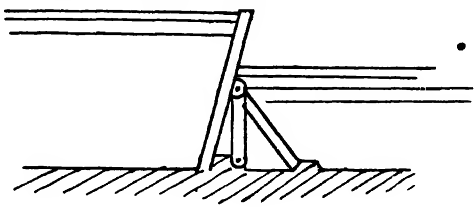

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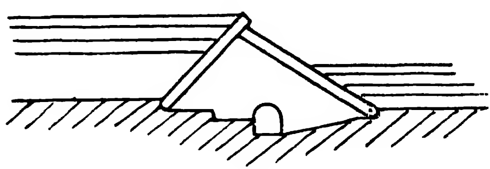

FIG. 137

through an opening in the side wall, and they are made to fall by placing this opening in communication with the downstream instead of the upstream reach. This kind of shutter is only suitable for passes of moderate width — say 100 feet — and it is rather expensive on account of the culverts [2].

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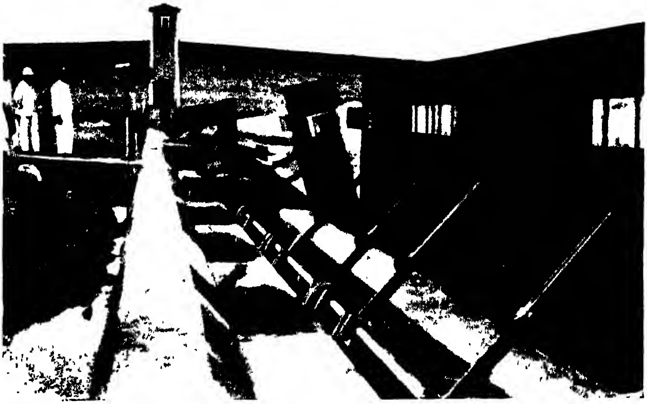
[1] *Rivers and Canals*, Harcourt *Trans. Am. Soc. C. E*, Vol 48, p. 3. The Paper gives details as to working and describes a modification called the Bebout wicket.

[2] *Proc. Inst. C. E*, Vol LX *Trans Am Soc. C. E*, Vol 48, p 3

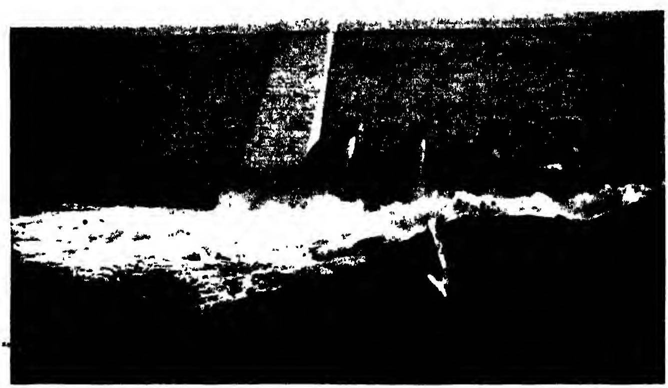
and the shutters are hinged to it. When the water rises to a certain height above its top, the shutter turns into a horizontal position, but as this causes a severe rush of water the shutter is usually raised by a chain attached to its lower end and worked from the bank. When in a horizontal position, it is held there by a ratchet. When the stream falls, the ratchet is released and the shutter is closed by the stream. This kind of shutter cannot be used where there is navigation.

On many of the weirs across the Indian rivers, the shutters are held up by a tie-rod on the upstream side. A trigger releases the rod. The shutters can be raised by means of a crane in the stern of a boat which is moored upstream of the weir and allowed to drop down. In other cases — including a barrage on the Nile — the shutters are made with back struts. The strut is hinged in the middle to form a knuckle joint. Water pumped into a ram causes the strut to collapse. Each group of 9 shutters contains a master shutter. When this is made to fall the other shutters of the group fall in succession.

Drum weirs, invented by Desfontaines, have been used in France and Germany. Two paddles (Fig. 139) are fixed on a horizontal axis and can turn through about 90°, the lower paddle, which should be slightly the larger, working in a "drum," which is roofed over and can, by means of sluices, be placed in communication with either the upper or lower reach of the stream. According as the upper paddle is to be raised or lowered, water is admitted from the upper reach above or below the lower paddle, the water on its other side being at the same time placed in communication with the lower reach. On the weirs first made on the Marne, the height of the upper paddle was 3 feet 7½ inches, and there were, in a weir, a number of pairs of paddles, each being 4 feet 11 inches wide. By having sluices at both abutments communicating with both reaches, and by opening or closing each of them more or less, the various paddles can be made to take up different positions, and thus perfect control over the discharge is obtained by simply turning a handle to control a sluice gate. A weir has since been made with a



Falling shutter, with back struts and master shutter. Ransomes & Rapier, Ltd., London and Ipswich.



Assuan Dam. View shewing discharge through upper sluices before masonry aprons were constructed (pp. 285 and 292).
Stoney Gates, Ransomes & Rapier, Ltd., London and Ipswich.

single pair of paddles extending right across the opening

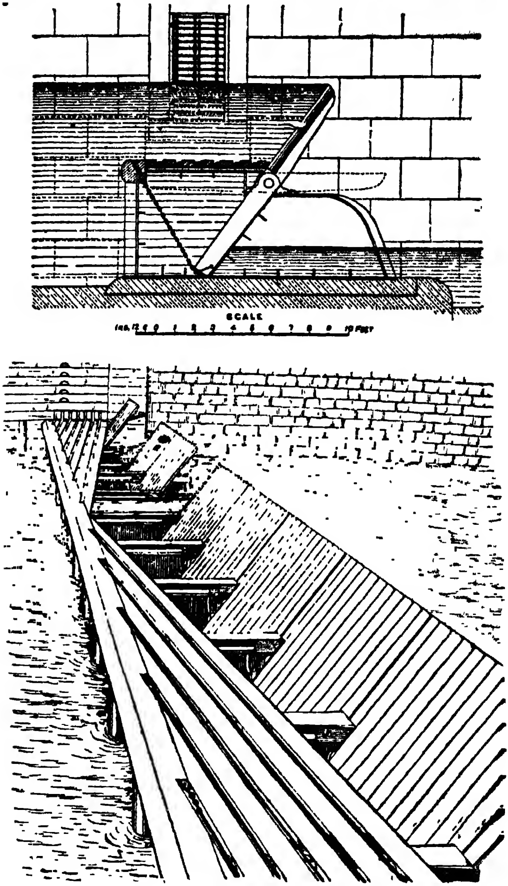


Fig. 138.

(33 feet), and the height of the upper paddle is over 9 feet [1].
The chief objection to drum weirs is the necessity for the

[1] *Proc. Inst. C. E.*, Vol. LXXXV

hollow or drum, which renders the work very expensive, except when only a small depth of water is held up.

The segmental gate or Taintor gate, turns (Fig. 140) on a horizontal pivot and works with very little friction. Gates of this kind were used on the old Nile barrages and were

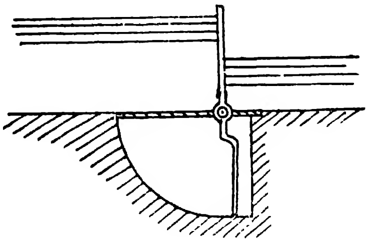
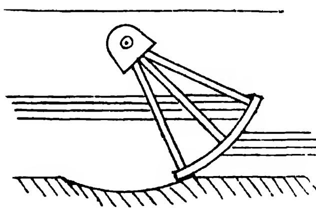


FIG. 139

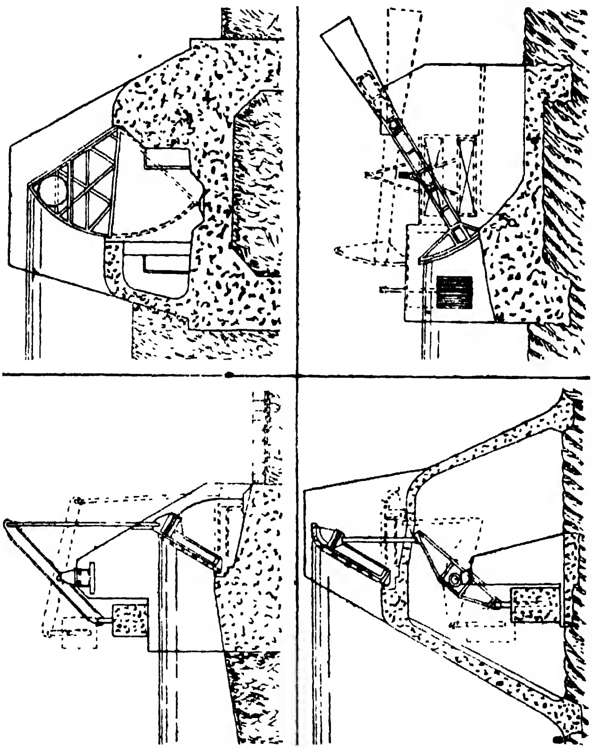
raised by chains. Fig 141 shows the gate on the crest of a weir into which it can descend Its height is regulated by means of a valve which admits water below the gate. The frame extending from the segment to the pivot can be pro-



longed beyond the pivot and can then carry a heavy counterweight (Fig. 142) The piers must be long enough to support the pivot. These gates are used with spans up to 100 feet.

There are also self-acting shutters which revolve on a horizontal axis at the lower edge, and are counterbalanced by weights suspended from an arm (Fig. 143). For a greater fall the weight can be inside the weir (Fig. 144). In another design the weights are cylindrical and roll on ways in the side walls, the grade of the ways varying so as to adjust the

resistance to the pressure as the gate falls. This arrangement is suitable when there in only one span, which may however be as great as 70 feet.



STAUWERKE A.G. ZÜRICH

FIG. 141, 142, 143 and 144

A weir used at Schweinfurt on the Maine, consists of a hollow iron cylinder, 59 feet long and 10 feet in diameter, running across the stream The cylinder is pear-shaped in cross-section, and can be made, by means of mechanism, to revolve, the water passing over it. Others of greater

lengths have been made. Another kind used at Mulhausen on the Rhine consists of a hollow wrought iron cylinder 85 feet long and 9.8 feet in diameter. It is open at the ends and the water enters it. The ends are in vertical chases in the piers. The whole cylinder can be raised by winches [1]. Instead of only one cylinder there can be 20 or more, of small diameter, one above another, the span being in one case 45 feet.

Art. 5. Bridges and Syphons. *Bridges.* — Only those parts of bridges are here considered which are exposed to the stream. If a bridge has piers there must be some disturbance of the water. The disturbance will be least when the area of the waterway of the bridge is at least as great as that of the stream, and when its shape is as nearly as possible the same. For small streams, a single span clearing the whole stream may be adopted, especially when the channel is of soft material but for a large stream the cost of intermediate piers, even with a certain amount of protection for them or with deep foundations, will be more than counterbalanced by the smaller thickness of arch or depth of girder.

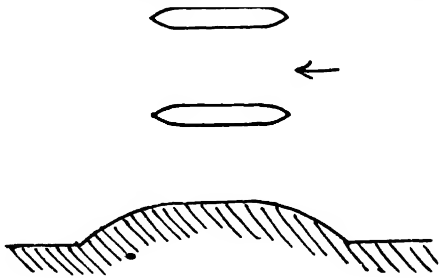
Generally a bridge has vertical abutments which limit the waterway, but it may have land-spans, and in this case the stream as it rises can spread out If the stream is of moderate size and is very slow, even in floods, it may be convenient to contract the waterway by making the distance between the abutments less than the mean width of the stream, but in most cases this is not desirable for the reason given above and usually the waterway is not reduced except by the piers. Piers and abutments should be so designed that abrupt changes in the section of the stream are, as far as possible, avoided, the piers being rounded or boat-shaped at both ends and the abutments suitably curved (Fig. 145). Boatshaped piers, besides presenting the neatest appearance, cause the least amount of disturbance.

A bridge can be made safe against scour either by giving deep foundations to the piers and abutments or by adding a floor and, if necessary, pitching. The former course is usually adopted and is the best. But in a case in which the

[1] *Proc. Inst. C E.*, Vol CLIII. and CLVI.

discharge of a stream of moderate size is to be increased or has been underestimated, it is often easier to add a floor to an existing bridge than to increase the span of the bridge (p. 224).

In any case in which the water rises above the crown of the arch, the bridge becomes a syphon, and a floor is probably necessary unless the foundations are very deep, or unless the rise of water above the crown is temporary.



On a canal the width between the abutments is generally made (Fig. 146) equal to the mean width of the stream if there are no piers. The arches, in Northern India, are usually of 60° as shown by the upper line. Arches of 90° as

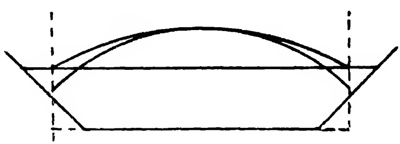


FIG. 146

shown by the lower curved line, have sometimes been adopted, the springing of the arch being below the full supply level, so that the stream is somewhat contracted. The 90° arch gives a reduced thickness and height of abutment, but it causes increased disturbance of the water, and this may necessitate downstream protection. An advantage in having the springing not lower than the full supply level is that this admits

of a raising of the full supply level in case the channel is remodelled.

Irrigation channels, especially the smaller ones, are very frequently at a high level, and bridges have ramps which are expensive to make and to maintain, and are incovenient. The lowering of distributary bridges in such cases, so that they become syphons, or nearly so, has often been advocated and is frequently desirable. The bed should slope down to the floor and up again The heading up can be recognised and shown on the longitudinal section. The crown of the arch can, if desirable, be kept above full supply level, so that floating rubbish will not accumulate.

Syphons and Culverts. — Syphons are used to pass drainage channels or other streams under canals or other lines of com-

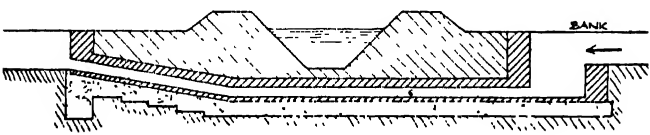


FIG 147

munication. In the case of a masonry syphon under a stream which may be dry while the syphon is full, the weight of the arch and its solid load must be not less than the upward pressure of the water passing through the syphon. The channel sometimes has a vertical drop at the upstream side (Fig. 147) and a slope at the downstream side. The slope enables any solid materials to be carried through, and facilitates cleaning out and unwatering. The drop at the upstream side does not give rise to any shock on the floor when the syphon is full, but a slope is preferable if there is room for it, and it causes less loss of head.

In large syphons it is usual to substitute a slope for the vertical drop. This was done in the case of the syphons, some of them very large (Fig. 148) for carrying torrents under the Upper Jhelum Canal (p 203). The velocities through the barrel are 15 to 20 feet per second. Economy in design was thus attained. The torrents only flow occasionally so

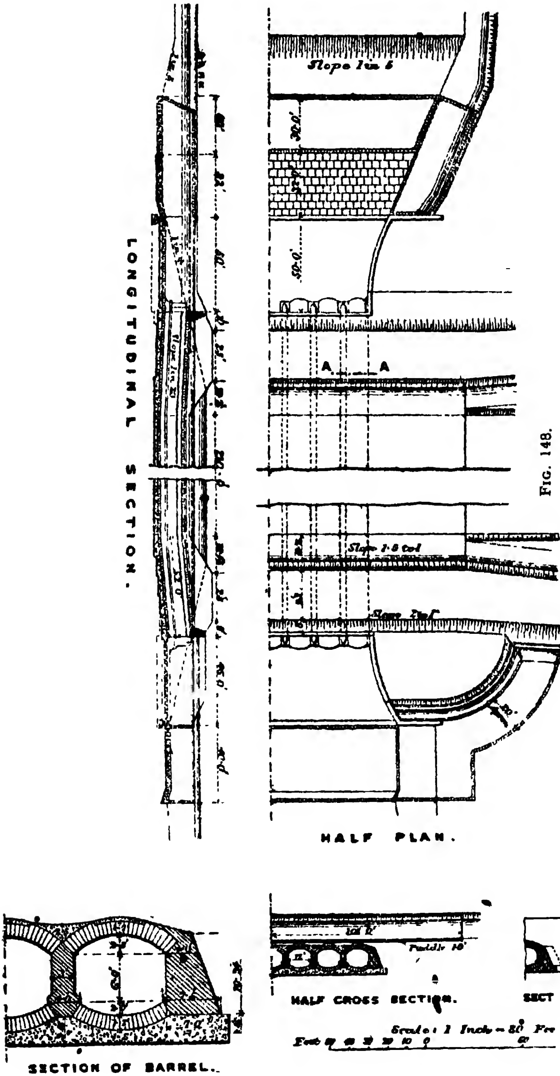
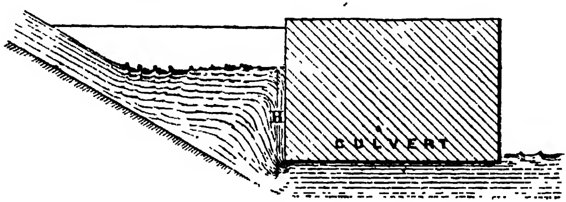

LONGITUDINAL SECTION.
HALF PLAN.
Slope 1 in 6
A A
SECTION OF BARREL.
HALF CROSS SECTION.
Puddle
SECT

FIG. 148.

that the question of wear does not arise [1]. In other cases the barrel of a syphon may consist of several lines of steel or reinforced concrete pipe. A long syphon for carrying the main drainage of New Orleans under a ship canal is made of concrete.

If the torrent becomes obstructed on the downstream side of a syphon the water on the upstream side may rise to a dangerous height. In the case of the Upper Jhelum torrents the downstream bed was kept at a height, above the floor of the barrel, of not more than two-thirds the height of the barrel.

It has been suggested that the wing walls of syphons should be so designed that it will not be necessary to alter



them if the canal is widened. In a syphon of the accompanying design (Fig. 148) it would be feasible to set back the canal banks a few feet by slightly raising and strengthening the walls which hold them up and adding short splayed walls in continuation of them. It would not, however, be very difficult to steepen the inside slopes of the canal, at the syphon, and line them with retaining walls which would curve into the bank thus forming a flume in which the velocity could be somewhat higher than in the rest of the canal.

Sometimes a canal or a torrent is carried over an aqueduct or "superpassage". The design is similar to that of a bridge with thick parapets to withstand the pressure of the water.

A culvert which is liable to run full and which has a steep approach channel (Fig. 149) may become suddenly drowned

[1] *Proc. Inst. C. E.*, Vol. CCI, p 39.

on the upstream side. As soon as the water rises to the crown of the arch, the wet border of the culvert increases and this reduces the velocity and discharge. The water coming down the approach channel then rises abruptly, and the increasep section of the stream causes a reduced velocity of approach, and this further reduces the discharge through the culvert. The heading up continues until the difference in the upstream and downstream water-levels is great enough to readjust matters [1]. The possibility of this heading up occurring should be attended to in the design. In the case of a culvert in a railway embankment where heavy floods have to be passed, the culvert may be made bellmouthed by a curved embankment constructed on its upstream side

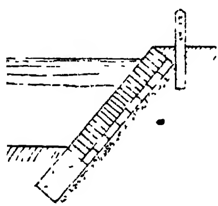


Fig. 150

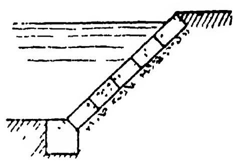


Fig. 151

Art. 6. General Notes on Works. *Pitching and Bed Protection* — Any scour upstream of a weir is generally due merely to the eddies formed upstream of it. It is not likely to be serious, unless a current parallel to the the weir has been set up (p. 251). And similarly as to scour upstream of a bridge pier or other structure. A hole formed alongside a pier or obstruction, if there is no floor, may however work upstream.

The object of pitching upstream of any work, or downstream of many bridges, is to protect the work from such scouring action as there may be. There may be pitching of the sides only, and it may be of loose stone (Fig. 29, p. 92) or of brick-on-edge laid dry and under this one brick flat resting on rammed ballast (Fig. 150). An alternative type of toe wall is shown in Fig. 151.

Immediately downstream of regulators, weirs or sluices

[1] *Proc. Inst. C. E.*, Vol. CLXXXVI

where there is great disturbance, both side and bed pitching may consist of blocks of concrete or of masonry (Fig. 152). Concrete is the heavier. Continuous slabs are liable to crack from settlement and require time to set. Further downstream — and downstream of bridges if contracted or having piers which cause a rush of water, especially if the soil is soft — the side pitching may be as in Fig. 150 but with the bricks over one-sixth of the area placed on end and projecting for

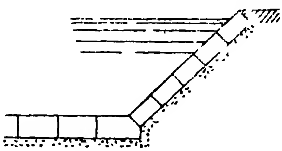


Fig. 152

half their length. This "roughened pitching" tends somewhat to reduce the eddying. The bed protection should still be blocks of concrete or of masonry. If a hole tends to form in the bed downstream of the pitching more blocks of masonry or concrete can be laid and left to take up their own positions

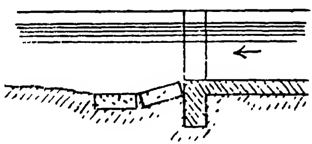


Fig 153

(Fig. 153). Sometimes crates filled with stone or brick bats are used.

The width between the side walls of a structure is likely to be greater or less than the normal bed width of the channel in the lower reach, according as the structure has or has not got piers. In the latter case the bed width can be restored immediately below the wing walls, and the pitching made accordingly. In the former case the toe of the pitching is not made to project beyond the side walls. The pitched length is made with a tapering bed width.

In some parts of the Punjab, large bricks, the length, breadth, and thickness being about twice the corresponding dimensions of an ordinary brick, are made, and are extremely useful and cheap for pitching. Where the soil is sandy such bricks can be burned without cracking.

It was once the custom in India to splay out the sides of a channel, downstream of a regulator or weir, so as to form a large pool in which the eddies exhausted themselves, but this gives curved banks and requires extra land and is not a convenient or neat arrangement.

Pitching has often to be replaced or extended owing to failure to pitch a sufficient length, to ram well the earth under the pitching, to use properly rammed ballast or flat brick, to give proper bed protection, or to the use of dry brick pitching when a stronger kind is needed.

The side slopes of pitching should be not steeper than 1 to 1. They can be ½ to 1 in rare cases, e. g., when there is no room for 1 to 1, or in continuation of existing ½ to 1 pitching. No rule can be laid down as to the length to be pitched. In a Punjab distributary it is often about 5 times the bed width.

Alterations — Alterations and remodellings of works on any channel may have to be undertaken. As far as possible they should be foreseen and allowed for. If the discharge over a weir has to be increased it may be feasible merely to raise the side walls. If only the water-level has to be altered this can be done by altering the crest level, or a subsidiary weir can be added. The waterway of a regulator can sometimes be increased by removing some piers and substituting needles for planks or gates. Alterations to syphons and bridges have been dealt with above.

General Remarks. — The floor of a regulator, sluice or barrage in a stream should usually be placed at a level somewhat lower than the mean bed-level of the stream. The bed may possibly be lowered in course of time. Lowering the floor also gives a greater thickness of water cushion to take the shock of water falling over the gates or planks.

If the structure is in a canal it is convenient to build, on the floor, a low wall or sill, reaching up to the level of the

bed or thereabouts, and running across from pier to pier under the line of gates or needles. The height of the gates or needles can thus be reduced, and there is little chance of silt or stones collecting and interfering with them. In the case of needles the wall must be strong enough to resist their horizontal pressure. If ever the bed is lowered the wall can easily be cut down or removed. But in the case of a barrage in a river, a sill is not so suitable because of the obstruction it would offer to floods. Allowing for the piers, the total area of water-way is generally about the same as that of the stream in its unobstructed condition. Where there is a sill it is less. The bed has to be heavily protected in any case. In the case of a wide channel, it is inconvenient to make the distance between the abutments of a work much less than the width of the channel.

There are of course very great differences in the styles of works. In the double regulator shown in Fig. 133 one set of needles comes close to the other. In Figs. 132 more room is left for working or temporarily stacking the planks. There are steps and 3 sets of grooves. In America where labour is extremely expensive reinforced concrete is largely used. All parts of structures so made are thinner, and the general appearance of the work is much affected. India and America represent two extremes. The former has cheap labour and the structures are often very substantial and sometimes are slightly embellished.

The principles of design are unvarying. A study of the design and history of a great weir will convey useful lessons even though a work of comparatively insignificant size is in contemplation or in hand.

Of the many kinds of apparatus described in this chapter each possesses some advantages and disadvantages. Gates require a bridge with lifting apparatus, and are suitable for large bodies of water and great depths. Comparing needles with beams or "stop-logs", the former can be worked by one man and admit of rapid removal, and require far fewer piers. They do not require hooks. Beams generally require two men, and are sometimes liable to jam, but obstruct floating rubbish less than needles. The falling water however, throws a

strain on the floor or necessitates a cistern. When all the water is to be diverted, beams, in shallow water, give rise to less leakage. The double grooves — if existing — enable earth filling to be used on occasion and leakage can then be quite stopped. For large works the advantages are generally with needles, but for small works and shallow water, with beams. Beams are more likely than needles to arrest rolling sand. Beams are very suitable for escape heads which have only occasionally to be opened, earth being filled in between the two sets of beams. Falling shutters of the Chanoine type admit of very rapid lowering, and can be used without a foot-bridge. The drum weir is perfect in action, but its cost is high.

At any system of sluices the regulation should be so arranged as to minimise the chances of damage to the bed and banks where this is at all likely to occur. If the gates are opened only near one side of the structure there will be a rush of water on that side, and serious damage may occur. The opening should be done symmetrically and, as far as possible, distributed along the whole length

Until experience has shown it to be unnecessary, soundings should be taken at regular periods of time below every important work where scour can occur. When scour is found to have occurred at any particular part of the work, the rush of water at such places should, as far as possible, be prevented, and a chance given for silting to occur. The Assuan dam is founded on rock but scour of the rock occurred below the sluices.

Unless experience shows that damage is not likely to occur, a stock of concrete blocks, sand-bags, or other suitable materials should be kept on the spot ready for use. Life-buoys should be provided on any work where large volumes of water are dealt with, especially if it is unfenced in any part, or if any of the men employed are casual workers.

Training Works. — The object of the upstream and downstream protections already described is to prevent damage to the structure owing to the disturbance caused by the structure itself. When a river is given to shifting its course

and cutting away its banks, protection of another kind is required. The stream, if left to itself, may cut away one bank upstream of the structure for a long distance, and eventually damage, or destroy by undermining, the upstream pitching and the abutment itself. This is known as "out-flanking." If in the neighbourhood of the line A B (Fig. 154) there is nothing for the river to damage — if, for instance, the structure is a weir with a canal, if any, only on the opposite bank of the river — and if the land is of no particular value, the case could conceivably be met by protecting the abutment on all sides, but even then there might be a chance of the erosion of the bank continuing until the stream had formed a connection at C with the downstream reach. This, of course, in the case of a weir, would render the work useless and might even destroy it.

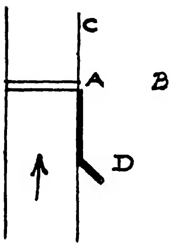


FIG. 154

In the case of a bridge carrying a road or railway, or of a syphon or aqueduct carrying a canal or other stream, it is wholly inadmissible to allow the stream to cut away even as far as the point A for fear of its severing the line of communication. Thus in every case it is practically necessary to prevent any serious erosion of the bank upstream of the structure. The bank DA can be protected by any of the methods given in Chap. V., Art. 3, (see especially p. 98), the protection being, if necessary, turned inwards, as shown at D, to prevent the end of it being damaged. See also p. 128.

The Bengal Dooars Railway runs near the foot of the Bhutan Himalayas, and crosses some broad river channels which, after the excessively heavy rains which occur, are filled by streams of very high velocities. One such channel or set of channels (Fig. 155), more than half a mile wide, is provided with a bridge having ten spans of 60 feet each. The railway embankment across the remainder of the channel having been breached in many places in 1903, protection was afforded by T-headed spurs and other groynes, the first arrangement, which withstood the floods of 1904, being as shown in the

figure. The triangular apex of the A-shaped groyne, south-east of the bridge, was added in 1905 because, in its absence, the water struck the bridge obliquely. After the addition there was a great deposit of silt in the neighbourhood of the four T-headed spurs. Next year the river, in a great flood, rose over the top of the railway embankment near these spurs and finally caused a breach 600 feet wide. The embankment was afterwards raised. The velocity through the bridge seems to have approached 18 feet per second. The bridge had at

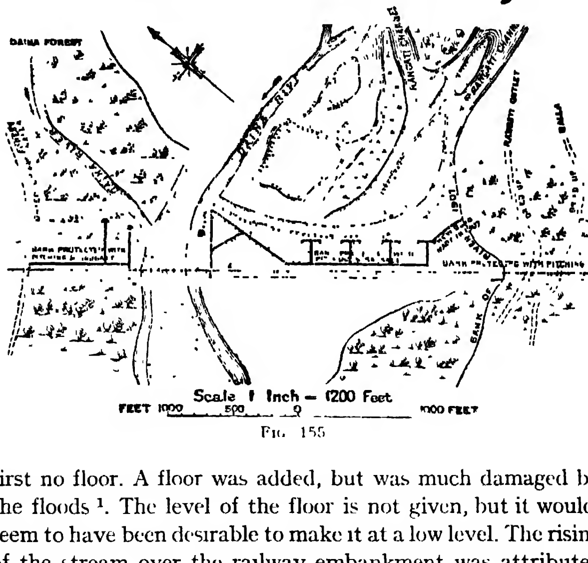


FIG. 155

first no floor. A floor was added, but was much damaged by the floods [1]. The level of the floor is not given, but it would, seem to have been desirable to make it at a low level. The rising of the stream over the railway embankment was attributed to the silting up near the T-headed spurs. It seems rather to have been due to a stream of high velocity being compelled to turn abruptly. The addition of the triangular apex made matters worse. With low velocities there would have been little trouble. If all the trouble could have been foreseen, it might have been best to build an additional bridge 2000 feet south-east of the existing bridge. The groynes were

[1] *Proc Inst C E*, Vol clxxx.)

composed of the wire-network rolls, described in Chap. V., (p. 97), piled pyramid fashion.

Notes on Sand

Classification of Sand. — In classifying sands the terms "coarse" and "fine" have no very definite meanings. Sands classed as "coarse" by various writers are generally from .01 inch to .03 inch in diameter but on the Irrawaddy the diameter in some cases reached .1 inch. An Indian and an Egyptian classification are given on p. 45, Bligh's classification on p. 244.

On p. 42 sands are classified according to the rate v, at which they sink in still water. Roughly $v = 10{,}000\, d^2$, where d is the diameter of the grain in centimeters[1]. According to this law, sands of classes. $.\bar{1}$ and $.\bar{2}$ are .007 inch, and .014 inch in diameter respectively but the specific gravities and shapes of grains of sand differ considerably and this of course affects the rate of sinking.

Percolation of Water through Sand. — The following laws were discovered by Hazen[2]. In mixed sand the water flows round the larger grains. The intervening spaces are occupied by smaller grains. The finer 10 per cent. have as much influence on the velocity of the water as the coarser 90 per cent. Thus the "effective size" of grain in any particular mixture, is such that 10 per cent. by weight of the material is of smaller grains and 90 per cent. is of larger grains.

At 50° Fah. and in clean sands of which the effective size is .10 to 3 millimetres,

$$V = 3{,}280\, d^2 S$$

where V is the velocity in feet per 24 hours, d is the effective size (diameter) as explained above, in millimetres and S is the hydraulic gradient. For any temperature t other than 50° the result of the formula must be multiplied by $\dfrac{t + 10.}{60}$

For sands which are not clean, V must be multiplied by .5

[1] *Proc. Inst C E*, Vol CCXII p 400

[2] *Report of Massachusetts State Board of Health* 1882, p. 541

to 7. For great exactness a "uniformity co-efficient" can be introduced. This is the ratio of the size of the grain which has 60 per cent. of the mixture finer than itself to that of the grain which has 10 per cent. finer than itself. V is divided by this uniformity co-efficient.

Hazen's formula can easily be used for approximate results, the effective size of the grains being estimated. For greater exactness specimens of the sand must be examined.

If the effective size of the grains exceeds 3 millimetres, V increases less rapidly than S. For coarse gravel V varies as $\sqrt{S}$ as in the case of flow in a pipe.

Other formulae have been devised by Slichter and Baldwin-Wiseman but in order to apply them, lengthy experiments have to be made on the soils

CHAPTER IX

Reservoirs and Dams

Art. 1. Reservoirs. *General Description.* — The object of a reservoir is to store water. The storage is effected in order to supply water for hydro-electric power, or for irrigation or navigation canals or town supply, or in order temporarily to get rid of flood water.

The water from a reservoir may be used in a very regular and steady manner, as for instance in town supply and in some cases of water-power and in some navigation canals. For irrigation it is likely to be used chiefly during certain seasons of the year, the quantity utilised daily increasing more or less steadily during some parts of the season, decreasing during others, and ceasing at times of heavy rain. In cases of flood storage the reservoir is kept as nearly as possible empty so as to leave the greatest possible amount of room for flood water. When the flood is over the reservoir is emptied as fast as the water can safely be disposed of. A reservoir generally depends for its supply on the yield of a particular valley or valleys which form its catchment area, and the capacity of the reservoir or reservoirs can be altered by altering the height or number or the dams. In cases of power or town supply the need for a large reservoir is owing to the inequality in the distribution of the rainfall. The reservoir is needed to "equalise" the flow — that is, to give a steady flow for an intermittent one. If the rain fell in equal quantities week by week, the daily fluctuations could be equalised by quite small reservoirs. The above remarks apply also to canals except that the intermittent supply has not so much to be equalised as to be made continuously available. In any case, the smaller the reservoir

the sooner it will go dry in a drought and the sooner it will overflow in wet weather and cause waste of the water. In other words, the larger the reservoir the better it will fulfil its function of equalising the flow and the greater the degree to which the catchment area will be utilised. The size is limited by considerations of cost.

A dam is generally of earth or masonry. The site of the dam is, whenever possible, selected at a place where the valley is narrow If the dam is made near the outlet of a natural lake the cost of the work may be small compared with the volume of water impounded. Notable examples are Loch Katrine and Thirlmere. The water-levels were raised 4 feet and 50 feet respectively The area of Loch Katrine is by far the greater. The sizes of these lake reservoirs are however far exceeded by some irrigation reservoirs in Egypt and America. A reservoir is usually provided wth a waste weir, also called a "spillway", over which flood water automatically flows when the reservoir is full.

Where there are several good sites it may be cheapest to have several reservoirs instead of only one. It is however likely to lead to a greater loss from evaporation and absorption, and to higher maintenance cost. Some of the reservoirs may be situated in side valleys that is on tributary streams; or there may be several reservoirs in one valley, one below another, the water from the upper ones passing through the others. In such a case it is desirable that the lower reservoirs shall be the biggest so that an accident to an upper dam may not wreck the others. In Madras where there are great numbers of irrigation "tanks", the dams of a few which were high up on a stream breached during a flood and several others lower done were in consequence wrecked.

Sometimes the water from a stream in one catchment area is diverted, by means of a channel passing though a tunnel or cutting, into an adjoining catchment and conducted to a reservoir in the latter. In such a case the channel is not usually able to carry the flood water and much of it has to run to waste.

In some cases a reservoir is formed in a natural depression and is supplied, not from the flow off a catchment, but from a river by means of a canal. In other cases the reservoir is formed by throwing a dam across a river. The great Assuan reservoir was formed by throwing a masonry dam across the Nile. It is provided with sluices through which the whole discharge of the river can pass. There is no waste weir.

The dam is nearly always of masonry — this includes concrete — or of earth. An earthen dam is cheaper and more quickly constructed than a masonry dam, and it can more easily be raised or strengthened at any subsequent date. An earthen dam is most suitable in a moist climate and where there is no rock for foundations. It is not feasible, however, in a case where there is no suitable earth wherewith to contsruct it. A masonry dam is suitable in a narrow rocky gorge and at any site where there is a sound rock foundation, not too far below the surface and where material for building the dam is easily obtainable, or where there is little room for a waste weir. In every case — but more especially when the valley is thickly populated — safety is the paramount consideration. Earthen dams have been constructed up to heights of about 100 feet. For higher dams masonry is used. Such dams are in rocky country

The question of losses of water from reservoirs has been dealt with in Chapter VI (p 142). The geological formation of the reservoir site must be fully investigated. Any strata whose edges are exposed to the floor or sides of the valley are likely to give rise to leakage. Porous material such as gravel or sand may render a site useless unless there is underlying rock, shale or clay. When the valley is covered with impervious material, patches of porous material or irregular cracks may give rise to serious leakage. In such cases a close examination of the whole area is necessary. Hard impervious rocks are generally fissured. The great majority of existing reservoirs are sufficiently water-tight. Their capacity for holding water tends to in-

crease as already stated. In order to ascertain the capacity of the reservoir, contour lines should be run round it, at levels about 5 feet apart. If the slopes are very steep the levels may be 10 feet apart.

In the case of a supply for a town there are, in addition to the impounding reservoir or reservoirs, "service reservoirs", the latter being of comparatively small size and their object being to store, near to the town, a supply sufficient for a short period.

In the British Isles, when the water of a stream is impounded, "compensation water" has to be given back to the stream lower down, for the benefit of riparian owners and mills. This compensation water is generally given in the form of a constant supply, and amounts to perhaps a quarter or a third of the available supply. It has to be included in calculating the daily supply taken out of the reservoir. The advantage to the stream in having this addition to it during dry weather is very great. Sometimes instead of a given proportion of the water being returned to the stream a certain proportion of the catchment area is set aside and a reservoir constructed for the compensation water.

A method of increasing the yield of a catchment by small drains or furrows has been mentioned (p. 38). In Western Australia a catchment area of 2,000 acres may have 200 miles of furrows They are made by the plough and discharge into larger collecting drains. The latter, if so steep as to cause scour, have their slopes flattened by means of weirs of planks, masonry or concrete [1].

Silt Deposit in Reservoirs — Regarding the deposit of silt in reservoirs very few exact figures are available. The reservoir of Lavagnina in Italy had once a capacity of 1,050,000 cubic metres. Silt deposit has reduced it to 640,000 cubic metres in the course of 20 years [2]. The proportion of silt to water in rivers varies very greatly both as to country and time of year (p. 44). The proportion is

[1] *Proc. Inst. C. E.*, Vol. CCV pp 366—367
[2] *Industria*, Vol 35, pp 196—201

greatest in floods and particularly in the earliest floods. It should of course be studied in cases of reservoir construction. A reservoir situated high up among hills receives less silt than one lower down. In reservoirs in the British Isles the silt deposits are insignificant. In India many old reservoirs of small capacity have become silted up and have been abandoned.

In the case of the Assuan dam the early Nile flood — that is the flood as long as it is rising — is allowed to pass through the sluices without any heading up by the gates. The falling flood is stored. If any silt deposits it is probably scoured out by the next season's rising flood. But the reservoir consists wholly of the river channel. In an ordinary reservoir no considerable scour can be obtained by means of sluices, except when the water-level is very low, and such scour as can be obtained involves waste of water. The scouring sluice has to be far larger than the ordinary outlet. Sometimes such sluices are provided in the Bombay irrigation reservoirs. The main object is not to scour out silt already deposited, but to pass off the earliest floods — and let them run to waste — and so prevent their depositing silt. To lay reservoirs dry and clear out the silt is impracticable.

It is possible to make a reservoir big enough to allow for a certain amount of silt deposit and even to contemplate its being completely filled up after a certain number of years, by which time it will have repaid the cost of its construction and it will be feasible to construct a new reservoir or reservoirs. This idea seems to have entered into the designing of some of the United States irrigation reservoirs mentioned below. In the case of the Bombay reservoirs, Strange has suggested that 10 per cent. be added to the calculated capacity of a reservoir to allow for silt deposit.

Afforestation of the catchment areas of reservoirs is useful in reducing the quantity of silt brought into them.

The Outlet and Waste Weir. — The "outlet" by which the water is drawn off from the reservoir, is placed at a low level and the flow from it is regulated by gates or valves.

The water below the level of the outlet sill is "dead" and cannot be drawn off and its volume is not included in the capacity of the reservoir. It serves to keep fish alive when the rest of the water has been drawn off.

The "full supply level" of the reservoir, AB (Fig. 156) corresponds with the crest of the waste weir. Under ordinary circumstances the water-level in the reservoir cannot rise appreciably higher than AB. It may of course be much lower. In the case of a flood the water-level rises to a "high flood level" CD. The level of the top of the dam has to be fixed with reference to CD. The greater the length of the waste weir the greater is its cost but the less is the height AC and the less the cost of the embankment.

Let A be the area, in square feet, of the water surface of the reservoir when the water-level is midway between

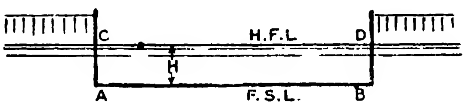


FIG 156

AB and CD. The volume AH is called the "flood absorption capacity" of the reservoir. When A is small, this volume is ignored and the weir is so designed that the discharge over it under the Head H is the same as the flood discharge Q — when at its maximum — of the stream entering the reservoir. This volume must in all cases be liberally estimated.

If A is great so that the volume AH is a considerable proportion of the total estimated volume of the heaviest flood, the weir can be provisionally designed as above and calculations then made to see whether it can be made smaller. The time which will be occupied in raising the water-level from AB to different levels must be calculated.

Let A be 25,000,000 square feet — nearly a square mile — and let Q be 1,000 cubic feet per second, continuing for 24 hours or 86,400 seconds, and let H be 3 feet. With

a free fall and $c = .7$ in the weir formula, the length of weir required to discharge Q is 51.25 feet.

For a head of	3.0	2.0	1.3	feet
q is	1,000	553	285	c. ft. per second.

At the end of 8 hours from the commencement of the flood the head on the weir would have been 1 foot if no water had run out. At the end of 12 hours it would have been 1.5 feet. Assume it to be 1.3 feet. The discharge at this stage is 285 cubic feet per second. The mean discharge over the weir while the head was increasing from zero to 1.3 feet was about half of the above figure or 143 cubic feet per second. The discharge being added to the reservoir was 857 cubic feet per second so that the rise in the water level in the 12 hours has been $1.5 \times \frac{857}{1,000}$ or 1.29 feet which practically agrees with the assumption made.

While the water-level rises from 1.3 to 2.0 feet the mean discharge over the weir is approximately $\frac{285 + 553}{2}$ or 419 cubic feet per second. The volume added to the reservoir is 581 cubic feet per second. This raises the water-level in 8 hours by $\frac{581}{1,000}$ feet or .58 ft. Thus the rise of .7 ft. occupies some 10 hours. If the discharge over the weir is now taken to remain steady at 553 cubic feet per second so that 447 cubic feet per second is added to the reservoir, the water-level rises only .11 ft. in 2 hours — .45 ft. in 8 hours — so that after 24 hours in all, the head on the weir is 2.11 feet. The flood is assumed to last only 24 hours. The greatest head on the weir will not exceed 2.11 feet and it can be designed accordingly, exact calculations being made.

If allowance is to be made for the flood continuing — with the same discharge — for several days, the flood absorption capacity of the reservoir may be all used up and the weir must be as at first designed.

In Chap. VII (p. 211) it is stated that the average discharge during the time t, occupied by the flood, is perhaps

$\frac{Q}{2}$. This is merely a general estimate. It can be adopted when the conditions suit it. In cases where floods are violent it is often preferable to consider the discharge Q to be constant as in the example just given. The time as well as the discharge must be liberally estimated. Any flood can be treated as of constant discharge if the time occupied is suitably adjusted. Hydrographs of the floods are of course most useful when calculations are made. Any water whch can be passed off by the outlet or by temporary escapes to high ground — these are used sometimes — can be allowed for.

When a reservoir is in use its capacity should be calculated for every few inches of depth, and the rise or fall of the water-level should be continuously recorded and also the discharge flowing through the outlet and over the waste weir, so that variations in the yield of the catchment may be noted and recorded.

Art. 2. Capacity of Reservoirs. The capacity of a reservoir should be just sufficient for its requirements.

The scale on which water is stored for power purposes varies (p. 154) very greatly. Whatever scale is adopted the capacity of the reservoir must be worked out to suit it. As in other cases above dealt with, if the reservoir capacity is stinted it will fail in a dry year, if lavish there is needless expense.

In the Tata hydro-electric works (p. 156.) the storage capacity of the Shirawta and Walwhan lakes is about 10,100 million cubic feet. The volume of water required at the forebay—during the annual 9-month period in which these lakes provide the water — is 6,700 million cubic feet. The excess capacity allows for additional storage in wet years for use in dry years.

The capacity of the "monsoon lake" is 14 days supply at the forebay. This lake is merely for the purpose of adjusting the inequalities of the daily monsoon rainfall. The total fall in the monsoon is ample for the requirements during the monsoon. The margin of 14 days might seem small in case for instance of the monsoon being late, but the monsoon usually

lasts for more than 3 months that is the 9-month supply above mentioned allows for a late monsoon.

Regarding the working of the reservoirs it was proposed, at the beginning of the 9-month period, to estimate the total draw-off from Walwhan and to lead in water from Shirawta to Walwhan only until the supply in the latter was rather more than would suffice for the rest of the 9-month period. Thus at the end of the period the bulk of the balance would be in Shirawta which is the higher lake, with a small emergency balance in Walwhan.

During the monsoon if Walwhan was spilling and the monsoon lake was not, the draw-off would be from Walwhan. The total water stored is a minimum just before the monsoon. In case of a partial failure of the monsoon the two main lakes may not fill. Shirawta water would not be led into Walwhan until it was seen that the latter was not likely to fill from its own catchment while Shirawta might do so.

In reservoirs for irrigation the capacities do not always bear any approximate ratio to the areas annually irrigated from them, or, what is roughly the same thing, to the volumes of water used annually. In India many of the older and smaller reservoirs are filled and emptied several times in an ordinary year. In a dry year there is either a great decrease in the irrigated area or a heavy failure of the crops. The modern principle is to make the capacity of the reservoir about equal to — or perhaps 10 per cent. more than [1] what can be replenished in a year of ordinary rainfall. Storing a greater quantity involves greater expense. In a dry year the irrigated area falls off but if the water is used with care the failures of crops are not likely to be great.

The capacity of one reservoir in Madras is about 2,500 million cubic feet, of one in Bombay about twice as much and of the Periyar reservoir in Travancore over 6,000 million cubic feet. In the United States the largest irrigation reservoirs — Pathfinder, Roosevelt and Elephant Butte— have capacities ranging from 45,000 million to 102,000 million cubic feet. These are from 2 to $3\frac{1}{3}$ years supply. The smaller

[1] *Irrigation, Roads & Buildings*, Strange.

reservoirs generally hold about one year's supply. The projected Hume reservoir on the river Murray in Australia will contain about 48,000 million cubic feet. The Assuan reservoir contains 85, 470 million cubic feet.

In the British Isles the distribution of the rainfall which is most trying for reservoirs occurs when the rain is heavy during the winter and light in summer. Fig. 157 shows a diagram for a reservoir in the "driest year". The distribution of the fall is supposed to be unfavourable as just described. The lower part of the figure shows the water-level at the end of each month, the reservoir being supposed to have vertical sides so that the quantity of water in it is proportional to the depth of water. The upper part of the figure shows the water

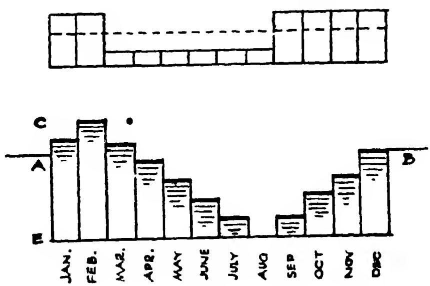


FIG 157

impounded (available fall multiplied by area of catchment) in full lines, and the consumption in a dotted line. The distance between the two lines in any month is the same as the rise or fall of the reservoir in that month There is supposed to be no overflow, and the total consumption of water in the year is equal to the quantity impounded in the year, so that the levels of the reservoir water surface on 1st January and 31st December, as shown by the horizontal lines A, B, are the same. Deacon, who has investigated the subject, has found [1] that, in order to satisfy the above conditions, the capacity of the reservoir must be 30 per cent. of the water impounded during the year, or about 110 days' consumption. On 1st January the

[1] *Ency. Brit*, Tenth Edition, Vol 33, "Water Supply"

reservoir must be about two-thirds full. At the end of February it is ready to overflow. At the end of August it is just becoming dry. The daily consumption is supposed to be steady throughout the year.

As an instance, suppose the catchment area to be 1000 acres and the mean annual fall 60 inches, with a loss from evaporation and absorption of 14 inches. The available rainfall of the year is (see last column of table below) 23.8 inches, or 1.983 feet. The water impounded and consumed during the year is $1,000 \times 43,560 \times 1.983 \times 6.25 = 539,962,000$ gallons. The reservoir capacity must be $\frac{3}{10}$ ths of this, or 161,988,600 gallons. This is represented by the height CE. If the mean yield of the rainfall in January and February is 6.3 inches, or .525 feet, the water impounded during those months is $1000 \times 43,560 \times .525 \times 6.25 = 143,931,000$ gallons, and the consumption is $\frac{539,962,000}{6} = 89,993,667$ gallons. The difference, 53,937,333 gallons, represents the addition A C, to the reservoir. Similarly, the light summer rainfall causes the depletion AE, and the heavy rainfall in the last four months of the year the addition E B. If the greatest height of the water-level above A B were less than A C, there would be overflow at the end of February; and if its greatest depth below A B were less than A E, the reservoir would go dry before the drought ended. If the capacity of the reservoir were increased either at the top or bottom, the cost would be increased and nothing would be gained so far, at least, as concerns the dry year under consideration. It is not meant that the highest and lowest levels of any reservoir designed as above would always, in the driest year, exactly correspond with the points of overflow and going dry, but they would do so generally. Deacon states that such a reservoir would fail about once in fifty years, and then only for a short time.

It might seem that a reservoir which would suffice for the driest year would suffice for any year. But it only suffices for the probably driest year. It is somewhat too small a reservoir. The rainfall of a very dry year might be (p. 12) only .50 of the average. The bigger the reservoir is the less is its

failure likely in any year. Moreover the reservoir considered above does not fully utilise the yield of the catchment area. In a wetter year there would be overflow and the yield from the reservoir would not be much increased. In a specially dry year the reservoir would fail. An increase in the capacity of the reservoir although, as has just been seen, of little use for the dry year, is exactly what is needed to make the reservoir more efficient. The capacity must be calculated for years of higher rainfall. The loss from overflow is thus greatly reduced.

By collecting information for large numbers of places in the British Isles, Deacon has prepared diagrams and tables which show the capacities and yields of reservoirs. In order to equalise the flow of the two driest years the capacity of the reservoir must be increased, its yield being also increased, and so on for larger groups of years. The following table gives the figures for the case where the rainfall is 60 inches and the loss by evaporation and absorption 14 inches: —

Number of Driest Consecutive Years the Flow of which is to be Equalised	Net Capacity of Reservoir for a Catchment Area of 1,000 acres	Daily Yield of Reservoir	Column 2 ÷ Column 3 or Number of Days' Supply contained in the Reservoir	Ratio of Rainfall to Mean Annual Fall	Available Rainfall
	Gallons	Gallons			Inches
1	166,000,000	1,475,000	113	.63	23.8
2	258,000,000	1,615,000	142	.72	29.2
3	329,000,000	1,987,000	165	.77	32.2
4	390,000,000	2,103,000	190	.80	34.0
5	441,000,000	2,187,000	201	.82	35.2
6	487,000,000	2,255,000	216	.835	36.1

The figures in the fifth column are those given on p. 13 except that .66 is there substituted for the older figure .63 used by Deacon.

The figures in the last column show the rainfall yields, after deducting the loss of 14 inches. It will be seen that, owing to this deduction, the yields for the shorter periods are reduced in a greater ratio than the figures in the fifth column.

In arranging for the supply of towns in the British Isles it is usual to design the reservoirs so as to equalise the flow of the three driest consecutive years. The number of days supply contained in the reservoir varies from 140 when the reservoir is in a wet district to 250 when it is in a dry one. On the average it is about 180 days. The above table shows that for the assumed fall of 60 inches and loss of 14 inches, the capacity of a reservoir, to allow for a six-year dry period, has to be 49 per cent. more than for a three-year dry period, while the daily supply from it is only 13 per cent. greater.

The following statement gives Deacon's figures for mean annual rainfalls ranging from 30 to 100 inches. The columns marked R show the reservoir capacities in millions of gallons, and those marked S the daily yields of the reservoirs in thousands of gallons. The figures for other falls can be interpolated. For a fall of, for instance, 50 inches, the figures, whether of R or S, are practically a mean between those for falls of 40 and 60 inches.

Number of Years whose Supply is to be Equalised.	F = 30		F = 40.		F = 60		F = 100	
	R	S	R	S	R	S	R	S
1	35	300	79	695	166	1475	345	3040
2	85	470	140	900	256	1815	495	3600
3	120	560	190	1050	329	1987	610	3900
4	150	620	230	1110	390	2103	710	4100
5	175	650	260	1170	441	2187	800	4230
6	195	680	290	1220	487	2255	887	4320

The number of days supply contained in the reservoir is as follows.

Number of years whose supply is to be equalised	Rainfall (Inches)			
	30	40	60	100
1	117	114	113	113
2	180	155	142	138
3	214	180	165	156
4	241	207	190	173
5	269	222	201	188
6	287	238	216	206

In the case of a low rainfall there is great variation in the figures in column 6 of the main table. Thus for a fall of 30 inches the first figure in column 6 would be only 4.9 inches. The figures lower down in the column are not so much affected. Hence the great increase — from 117 to 287 —in the number of days supply shown above as compared with that in the right hand column.

In all cases the loss is supposed to be 14 inches annually. If it is 15 or 13 inches, the reservoir capacity is less or more by about five, ten, or fifteen million gallons, according as the number of years in column 1 is 1, 3, or 6. And the daily yield in less or more by about 50,000 gallons.

With a low rainfall the advantage of a large reservoir is somewhat increased. The capacity of the six-year reservoir for a fall of 30 inches is 63 per cent. more than that of the three-year reservoir, but the supply is 22 per cent. greater.

In the article above quoted it is shown that if, as commonly happens, the consumption of water is in summer greater than the mean, and in winter less, the conditions are still more trying for the reservoir and that in the case where the summer consumption is 13 per cent. greater than the mean, the capacity of the reservoir which impounds the water of the driest year must be 33 per cent., instead of 30 per cent., of the total supply impounded during the year. It would then contain 121 days' instead of 110 days' mean supply. The tables from which extracts have been given are calculated on the basis of a constant consumption. This, however, in the case where the number of years whose supply is equalised is greater than one, makes, owing to the increased size of the reservoir, no practical difference.

The calculations for the great reservoirs in Radnorshire for the supply of the city of Birmingham are as follows[1]. The ratio of the mean fall in the three driest years to the mean annual fall was taken as .80 instead of .77. There is some difference of opinion as to the best figure:—

[1] *Proc. Inst. C. E*, Vol clxxx.

Mean annual fall determined from readings of various gauges	65 inches
Mean fall of three driest years	52 „
Deduct loss from evaporation and absorption and losses during floods	15 „
Available rainfall	37 „

This multiplied by 44,000 acres, the area of the catchment, gives 102 million gallons per day. Of this 27,000,000 gallons is compensation water, leaving 75,000,000 gallons for Birmingham. Capacity of reservoirs, 17,250,000,000 gallons, or 169 days' supply.

The figures given above for reservoir capacities are suitable for the British Isles. They assume, as already istated, that the distribution of the rainfall is the least favourable that is at all likely to occur. Deacon states that the figures do not relieve the engineer of the exercise of judgment. The chief questions on which judgment has to be exercised are whether to equalise the flow of three years or another number, and how much to allow for loss. As regards the British Isles, as already stated, three years is the period usually taken. The figures are suitable for most places in Europe, but in some places, for instance on the Mediterranean coast, the distribution of the rainfall is somewhat less favourable than in the British Isles. In other parts of the world, and notably in or near the tropics, the distribution of the rainfall must be specially studied. In any case which may arise and for whatever purpose the water is required the minimum reservoir capacity can be arrived at by drawing a mass diagram like that shewn in Fig. 158. The abscissae are times and the ordinates percentages of the yearly increment to the reservoir, the full thick line representing the fluctuating additions and the thin line the steady draw-off. BE is parallel to AD. At first the reservoir surface rises. At the end of February the net addition is AB and the reservoir is full. From the end of February to the end of August the water drawn off is KE. The water added is KC. The net depletion is GE and this is the minimum storage necessary. The diagram represents the worst conditions that are likely to occur in England.

The diagram can be extended so as to cover two or three years. If a long record of rainfall exists the diagram can be made to cover it all. Otherwise assumptions must be made, based on such knowledge as exists. In any case the reservoir capacity can, when safety from going dry is essential, be

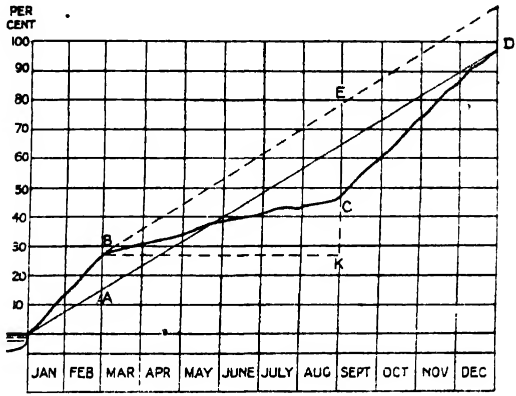


FIG 158

determined for what seem to be the most unfavourable conditions.In hot countries loss by evaporation from the surface of the reservoir should be allowed for.

When water is drawn for storage from a snow-fed river the necessary capacity of the reservoir is probably less than in other cases because of the smaller fluctuation in the river discharge.

Art. 3. Earthen Dams. *General* — The banks of artificial channels in earth and also flood embankments have been dealt with above (p. 220 and references there given) and the methods adopted to ensure good earthwork described. The dangers are the occurrence of piping or slipping. In earthen reservoir dams the dangers are the same but they are greater, the depth of water held up being generally much greater.

A fairly typical cross-section of an earthen reservoir dam,

with the reservoir full, is shown in Fig. 159. Owing to the flat upstream slope the water pressure is chiefly downward and there is no possibility of the dam being pushed along horizontally. The upstream slope is pitched and cannot be washed down but the pitching is not impervious. The gradient of the dotted line is 1 in 3.4. The outer portion of a

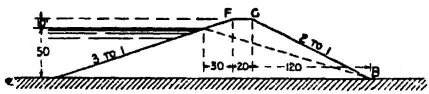
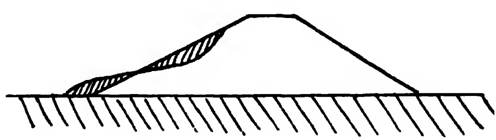


FIG. 159

high embankment sometimes slips (Fig. 160) and precautions should be taken against this. A slip may occur if there is unequal settlement owing to the work having been done at different times. It may occur during the construction of the work. One cause of slips is sudden and partial changes in the degree of saturation and another cause is excessive saturation. These may occur under the influence of the weather or they may be due to springs. Springs may develop after the reservoir is filled. Some clays when wet require extremely flat side slopes, and will not stand even at 5 to 1. The earth on the side next the reservoir is not likely to slip. It becomes



soaked, but it has the pressure of the water against it and, it is pitched. In Madras, where old reservoirs are very numerous, the slope on the side next the water is generally only $1\frac{1}{2}$ to 1. Slips do not occur if the side slope is made sufficiently flat. On account of its tendency to slip, pure clay should not be used in a dam except for the core wall.

The best foundation for an earthen dam is sound unfissured rock or compact clay The next is good firm earth.

Rock containing fissures is not satisfactory. Sand and porous strata are unsuitable.

When the site of an earthen dam has been provisionally decided upon — or when alternative sites are under consideration — the soil is examined by borings. These are if possible continued — especially along the line of the puddle wall —until an impervious stratum has been found. Faults in the strata may give rise to much uncertainty and necessitate prolonged investigations. The most suitable site depends to some extent on the type of design — as regards the depth of the puddle trench — which it is the practice to adopt. These types will be described directly.

Before the dam is constructed all the loose surface soil, vegetation and roots are cleared away. The soil thus exposed

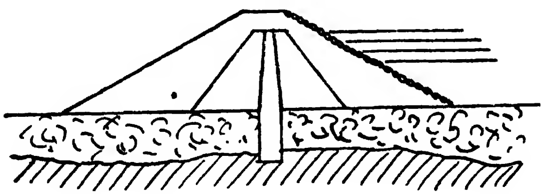


Fig 161

is then harrowed in a direction parallel to the length of the dam. The dam is then built. It has to be built of such materials as can be obtained near the site. The whole of each layer schould be constructed at the same time. In each layer every precaution should be taken, and the earthwork be of the very best.

An earthen dam has, in very many cases, a core wall — in the central part (Fig. 161) — which is generally of clay puddle. In England it is made in every case. It is carried down to an impervious stratum, whether this is at the surface of the ground or — occasionally — 100 or even 200 feet below it, and is keyed into it to a depth of a foot or more in the case of hard rock and several feet in the case of clay. On this core-wall the impermeability of the dam largely depends. The core wall may be of clay puddle, concrete or

masonry. Its top is horizontal and about level with the highest water-level. It is desirable not to make the foundation stepped, but to let it follow the profile of the impervious stratum. The wall is keyed at its ends into the sides of the valley or gorge.

A core-wall of concrete or masonry is, in a high dam, necessarily a comparatively thin structure, and it may be subjected to great strains by unequal pressures of the earth which surrounds it. It is therefore to some extent liable to crack. A concrete or masonry core-wall is most suitable when it is founded on rock. There are many core-walls of concrete in the United States. One built for the water-works of Boston is 100 feet high, 8 feet thick at the base, and 4 feet thick at the top. A clay-puddle wall, being plastic and moist, at least during the period immediately succeeding the construction of the work, is not very liable to crack. The top width of a puddle wall may be 4 to 10 feet, and the batter of the sides from 1 in 20 to 1 in 8. Sometimes the bottom width is made a third of the height. The clay used for the wall above the ground level should contain about 33 per cent of sand and stones. This diminishes its shrinkage when it dries. It should not be given too much water in mixing. It should be thoroughly mixed, worked up and trodden down. In England the puddle trench is, as already mentioned, sometimes carried to great depths. Clay and shale are not trusted as impervious strata if below them are sand, gravel or shingle. Drift clay is not trusted if it contains layers of sand and gravel. When hard rock is struck the trench is taken well into it to get clear of fissures.

In the British type of dam the portion nearest the core-wall on either side (Fig. 161) is generally made of earth specially selected for impermeability. The distance to which it extends from the wall depends partly on the quantity of such earth available. In any case it has to be very carefully made and consolidated, to avoid unequal pressures on the core-wall, or unequal settlement which might cause it to part from the wall. One of its functions is to keep the core-wall moist when the water-level in the reservoir falls. This filling in the

central part of the dam thus constitutes, together with the puddle core-wall, the main staunching part of the dam. The rest of the dam — the outer portions — are generally of material which is not so impervious.

Regarding the downstream portion of the dam most engineers drain it to some extent so as to reduce the chance of slips. The drains are cut in the surface of the ground and are filled with loose stone and gravel. The drains obviously tend to flatten the hydraulic gradient (p. 245). Some engineers do not use them If the central and upstream parts of the dam are of excellent quality and of ample dimensions but the downstream part is considered to be somewhat liable to become saturated and slip, drains are desirable. They are more required in countries where rainfall is excessive at certain seasons than where it is not so.

In America, core-walls of the kind described above, that is passing through the dam itself, are seldom made. The upper soil is cleared away, especially in the centre of the dam, till very firm earth is found and the dam is then built upon it. If all the material available is of good quality it is used throughout the dam, but generally the most impervious material is used in the upstream portion of the work, more pervious material in the middle and the most pervious in the downstream portion. Sometimes drains are used as in England. If good material for the downstream half of the dam is deficient coarse material or even gravel is sometimes used for it.

When the good material is decidedly scarce it is sometimes used for a core-wall in the centre of the dam, extending up to above the water-level There is often difficulty in obtaining clay of good quality for core-walls. It has been argued that a core-wall causes saturation of the upstream half of the dam and is a hindrance to the consolidation of the earthwork of the dam as a whole. This idea seems to be untenable. Slips do not occur in the upstream half of the dam if it has a suitable side slope. If a core-wall prevents water from reaching the downstream half of the dam it tends to prevent the occurrence of slips there.

When the upper strata of the soil are decidedly permeable, for instance gravel or sand or even disintegrated or badly fissured rock — especially if these strata are thick — one or more cores are made (Fig. 162) to join the dam to an impervious stratum or to rock. If the strata met with at a reasonable depth are not very impervious it is usual to give flatter slopes to the dam so as to increase its width. This is also done in cases where the material of which the dam is built is not very impervious.

In India also a core-wall of the British type extending through the dam up to above the water level is now not often used. Good clay for puddle is not easily obtained. If an impervious stratum exists at no great depth a core-wall, as in American practice, is taken down to it. In other cases there may be no impervious stratum except at a great depth. A core-wall is then carried down to a depth equal to half the

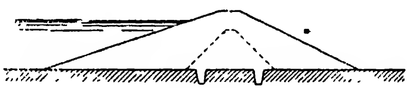


FIG 162.

depth of water in the reservoir if the soil is compact and to three-fourths of such depth if only fairly compact.

The most impervious materials obtainable are generally used for the central parts of the dam, the outer parts being of coarser material. Much attention is given to drainage as is fitting in a country where there are excessive monsoon falls.

Whenever, in any country, a central core-wall is used — whether or not it extends up through the dam — the best material should be used in the central part of the dam. It should be used in the upstream portion when there is no core-wall, that is when there is no porous material beneath the dam.

Failures of carefully constructed earthen reservoir dams are rare. They are rarer in England than in other countries.

Some Details. — The height of an earthen dam above high flood level is never less than 5 feet and may be 10 or 12 feet.

The top width is not less than 10 feet and may be 20 or 30 feet. The outer slope is generally turfed.

If the ground has a side-long slope it should be cut into steps as shewn in Fig. 163.

Probably the best and most water-tight earth is a mixture of materials of various grades from coarse and hard to fine and soft.

When a layer of permeable material extends under only the upstream half of the base of a dam it is sometimes covered over by a layer of puddle [1] which is joined to a vertical wall of the same — at the toe of the slope — and carried down to the impervious layer below.

Sometimes instead of a central puddle wall a layer of puddle is laid on the upstream slope of the dam. It is protected

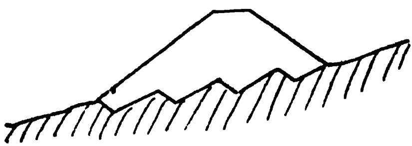


FIG 163

by pitching but is liable to be damaged by crayfish or the like and to dry and crack when the reservoir is laid dry. It is ineffective if there is porous material beneath the dam. If concrete is substituted for puddle it cracks owing to settlement of the earthwork Unless the thickness of the puddle is small its volume is far greater than that of a vertical wall carried to a moderate depth.

In order to afford full protection against waves and their splashes, the pitching on the upstream slope of a dam should extend up to a height of 5 feet, measured vertically, above the highest water-level. In the case of a dam in which the "fetch" or distance over which the waves have been in process of formation, exceeds two miles, the above height should be slightly increased [2]. The pitching is usually of

[1] *Rainfall, Reservoirs and Water Supply*, Binnie

[2] The height of a wave is supposed to be $1 \cdot 4\sqrt{\text{fetch}}$, but this allows nothing for splashing.

stones roughly squared at their outer ends and laid on a layer of broken stones.

While a dam is in course of construction arrangements must be made to deal with flood water. Generally the construction of some part of the dam has to be deferred to let the water pass. In the case of a masonry dam it does not much matter what part is thus deferred provided the usual procedure of stepping the work back is followed. In the case of an earthen dam it is best to defer a portion, not in the lowest ground where the dam is highest, but to one side of it, thus allowing the highest part of the dam to be brought up continuously. Temporary embankments and weirs can be constructed to cause the water to traverse the desired route without doing damage. Stepping of the earthwork should be avoided as far as possible. If it has to be adopted the steps should be small. Sometimes the flood water is conveyed away by means of a "by-wash", by an entirely different route.

In sinking a puddle trench any water met with is pumped out. The pump may be fixed in the deepest part of the trench at the downstream side. To it the water of any springs met with should be conducted by means of small pipes which can be covered over with concrete so that the trench is kept dry, the pumping being continued while the trench is being filled with puddle and the water-level kept below that of the top of the puddle. When the puddle rises above the water-level the pipe can be plugged with concrete. If the spring is very deep seated the pipe can be led to the lower side of the embankment [1]. Where the puddle trench crosses the stream of the valley the wall can be of masonry or concrete instead of puddle.

When leakage occurs in a dam it is serious if the water comes out muddy but in any case it should be dealt with in the manner already described (p. 191).

The Outlet. — The water from a reservoir when it is to be supplied under pressure, is usually drawn off by means of pipes which are laid inside a culvert of masonry or concrete.

[1] *Rainfall, Reservoirs and Water Supply*, Binnie

The pipes can thus be inspected. The culvert is blocked at its upstream end by a thick masonry or concrete wall through which the pipes pass. When the water is not to be supplied under much pressure the pipes are omitted and the water flows through the culvert.

The culvert should be taken, if possible through a tunnel, round the flank of the dam or across a saddle. When this is not possible it should be taken through a trench. If possible it should be founded on rock or on a hard impermeable stratum. The sides and bottom of the trench should be notched at intervals and the masonry or concrete should have projecting rings to fit into them. The whole should fit the trench closely. The rings should extend also over the top of the culvert. The earth filling should be extremely well consolidated.

Sometimes the culvert is taken — with precautions similar to the above — through the dam. This is not so safe. If there is a deep puddle trench, the puddle under the culvert, where it crosses, may settle down and part from it. Any unequal settlement of puddle and earthwork throws great strain on the culvert and may cause it to crack. The danger is much less when the core-wall is of masonry or concrete and is fairly thick. It can be thickened where the culvert crosses it.

At the upstream end of the culvert there is a masonry tower — access to it is obtained by a foot-bridge — and from it the gates or valves for opening and closing the culvert, or the pipes in it, are worked. If the reservoir is for the water supply of a town, it is arranged, by means of a vertical pipe, that the draw-off can be at various levels so that the surface-water can always be used In the case of some of the towers at the reservoirs whence Birmingham is now supplied, the vertical pipe consists of a number of steel cylinders with gun-metal faces which are so accurately made that the joint is water-tight when one cylinder merely stands on another. The draw-off is obtained from a given level by lifting a particular number of cylinders. Sometimes the tower is made of reinforced concrete. When it is lofty

it should be strong enough to resist a strong wind, blowing when the reservoir is empty.

If the core-wall is of masonry or concrete the tower can be built so as to be in one piece with it. No bridge is needed. Any portion of the culvert extending upstream of the tower is — when the culvert is closed — subject to the full water pressure inside it. Such a portion can be made of reinforced concrete or it can be omitted and long wing walls built, the dam near the outlet thus consisting of a masonry dam of less than the usual thickness but with earth behind it. In some cases, especially where the outlet is of large size and is used as a scouring sluice or for passing off silt-laden floods [1] (p. 294), the dam near the outlet is a masonry dam of the proper thickness — fitted in between two lengths of earthen embankment. The ends of the latter of course have wing walls.

In a few cases the outlet gates have been made sloping and have been worked by rods running up the slope of the dam. Such rods have to be of great length and to be supported at frequent intervals. In fixing the site of the outlet due regard is of course had to the alignment of the canal. A bad site is on the steep side-long ground where the dam crosses the stream. Here unequal settlement of the earthwork and great strains or the culvert are likely to occur [2].

The outlet should be large enough to give the full supply to the canal under the minimum head which will occur. It is then also capable of quickly drawing off the water from the reservoir — in case of need — down to a low level.

If the outlet sill is to be made near to the bed of the reservoir it can be made either quite close to it or somewhat above it. Even if considerable silt deposit in the reservoir occurs the outlet sill will not become covered by silt if there is a high velocity through the outlet.

Outlets may be a great depth below the water surface. Much power is then required in order to operate the gates especially when the gate is from 35 to 75 per cent. open.

[1] The drawing off of silt or rolled material may cause a deposit in the canal but any such deposit can easily be cleared out

[2] *Indian Engineering*, 3rd December 1921

This is due to the extra pressure set up by the flow through the outlet. Great vibration of the gate also occurs and it may cause rapid wear or loosen the frame of the gate from the masonry. The remedy is — when practicable — to keep the gate either open or closed and to make the opening bell-mouthed. To reduce the above troubles a new kind of gate with rollers — on trunnions fixed to the gate — has been adopted on a reservoir in Guatemala [1]. In America a needle valve is sometimes used under great heads. It acts in the same manner as the needle in a carburettor and the water pressure is used to operate it.

The Waste Weir. — The waste weir is frequently placed so as to be a continuation of the line of the dam where the latter is getting into high ground at the side of the valley. Where heavy floods occur, the side walls of the weir should be some 2 feet above the high flood level. The design can be based on Fig. 107, p. 236, where see also as to rounded crests. A still better site is on a saddle. In either case the discharging channel can have a slope more or less steep according to circumstances and be designed as explained in Chap. VI (see for instance pp. 132, 138, 178 and 186—191). If the weir can have a free overfall instead of being drowned its length and cost will be reduced. It can have a cistern (Fig. 132 p. 264). When there is a very steep fall in the ground there can — if necessary — be a series of weirs.

A long waste weir is expensive and a short one with a high value of H gives a lower full supply level besides causing heavy action on the floor. In the Bombay Presidency where much attention has been given to irrigation reservoirs and their working, it is becoming usual, owing to the above difficulties, to place hinged shutters along the crest of the waste weir, or to lower the crest of the weir over a certain proportion of its length and to fit this lower portion with large balanced gates on rollers. Some of these open automatically when the water against them rises to a certain height. Some of them travel downwards in order to open.

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[1] *Verein deutscher Ingenieure, Zeitschrift*, Vol 67 p 490 For brief description see *Engineering Abstracts, Inst C E*, No 17. Oct 1923

Overhead there are travelling winches [1]. In some cases the automatic gates are of a special design — by Sir. M. Visvasvarya — in which two gates open together, one moving upwards and the other downwards.

In places where there is a regular flood season the supply level is "restricted", before the arrival of the floods, to a certain level lower than that of the weir crest.

It is again restricted, when a flood has passed. Later it is gradually raised. About the close of the flood season it is raised up to the level of the top of the shutters and the volume of storage is thus considerably increased. The gates and shutters require skilled workers and skilled supervision but only for short periods.

The gates allow the heavily silted early floods to be quickly passed on. They and the shutters permit of a shortened weir. The arrangement suits the usual changes in the ground levels the gates being placed where the ground is low. It is most suited to a reservoir which is almost certain to be filled every year. In other cases the gates and shutters would be of no use in many years [2].

In Indian reservoirs the waste weir is sometimes in the position shown in Fig. 164, *ac* being the weir. In such a case a special hydraulic problem arises. In a case where a stream whose velocity is V issues from a reservoir or takes off at right angles from a larger stream there is [3] a fall in the water surface of about $\frac{V^2}{2g}$. The same thing occurs downstream of a weir, at least when there is a clear fall which is vertical or nearly so, so that the water after falling has no horizontal velocity. The water has to be started afresh on its course. In the case represented by the figure, the width of the channel is often restricted because of high ground beyond *f*, and the velocity in the channel may be very high. Suppose the channel below *cf* to be of masonry with vertical sides, and to have a 20-foot bed, a slope of 1 in 500, and a depth of

[1] *Reservoirs with Earthen Dams*, Strange.

[2] *Indian Engineering*, 1st October 1921

[3] *Hydraulics*, Chap II, Arts. 19 and 20).

water of 10 feet. The velocity may be 15 feet per second, and $\frac{V^2}{2g}$ is 3.49 feet. If the water has a clear fall over the weir at *e*, allowance must be made for a depth of water of 13.49 feet, not 10 feet, in the channel at *e*. Ordinarily the length *a e* will be much greater, relatively to *e f*, than shown in the figure. Suppose that *a e* is 300 feet and that the slope of the floor of the channel is carried on at 1 in 500 from *e f* up to *a*, *b*, *c*, and *d*, following in each case the lines marked on the figure which represent the directions of flow. The length *f a* will be about 310 feet, and the floor level at *a* will

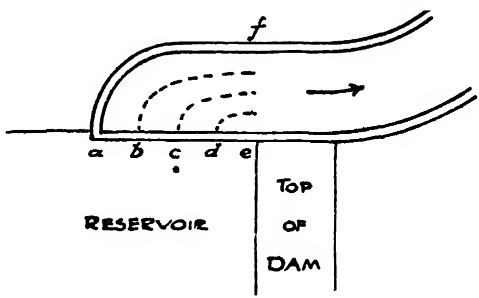


FIG 164

be about .62 feet higher than at *e f*. The water-levels below the weir will be in each case 13.49 feet above the floor. This should be allowed for in the design. It is true that the stream on first starting into horizontal motion below the weir moves more or less at right angles to it, and has thus a large sectional area and a low velocity, but it very soon has to turn parallel to the weir and acquire the full velocity of 15 feet per second, and there must be the requisite extra head to give this velocity. If the weir is drowned, the water on passing over it may have a high horizontal velocity, but it will be at right angles to the axis of the channel, and its effect will be wasted in eddies.

Art. 4. Masonry Dams. *Stresses and General Design.*—In a masonry dam, although the work is, or should be, of het

best quality, it is a rule to calculate the dimensions so as to give no tension on any part of the structure. The term masonry includes concrete.

Fig. 165 shows the upper part of a masonry dam. The lines with the arrows show the vertical force due to the weight of the masonry above A B — acting through its centre of gravity G — the horizontal force due to the waterpressure on it, acting at two-thirds of the depth, and the resultant of these two. In order that there may be no tension on the masonry, the resultant must always fall within the middle third of the

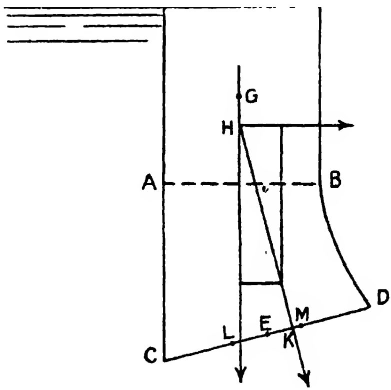
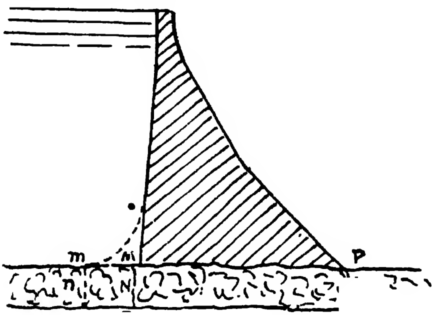


Fig. 165

thickness A B, of the dam. In order to prevent its falling outside the middle third — at a lower level — the downstream face must be splayed out, and the splay will go on increasing somewhat. Suppose now, that the reservoir is laid dry. It will be found that in the case of a dam more than 100 feet high the pressure due to the weight of the dam will fall outside the middle third — to the upstream side of it of course — of the thickness of the dam, and a slight batter must be given to the upstream face. The pressure of the water acts normally to the surface of the dam whether it is vertical or not. The admissible limit of pressure may eventually be reached

owing to the height of the dam — when both sides are vertical it may be reached in a wall about 220 feet high — and additional splay may have to be given for this. By following the above procedure the section of the dam can be calculated, beginning from the top and working downwards. The resulting profile of the dam is somewhat as shown in Fig. 166. If a masonry dam is designed on the principles given above — that is, so as to be safe as regards crushing and overturning — it will be safe as regards shearing horizontally, but test calculations can easily be made for this.



Calculations of the above kind do not, of course, enable all the stresses in a solid mass of masonry to be found. Great stresses are caused by expansion and contraction owing to changes in temperature. The method of calculation described above indicates a suitable form for the profile of a dam. The large factor of safety usually adopted allows for other stresses. The sections of the oldest dams, made in Spain, were somewhat as shown in Fig. 167, and contained about twice as much material as was necessary. The object of the calculations is to save this needless expenditure.

The maximum stresses are in a plane CD normal to the resultant HK. If the resultant passes through E, midway between C and D, the stress P is uniform throughout the section CD. If it passes through the "third point" M the stress is 2 P at D and zero at C.

If m is the specific gravity of the masonry the lower part of the downstream slope tends to assume the value 1 to $\sqrt{m}$ or approximately — in most cases — 1 to 1.5 or 1 to 1.6.

The safe shearing stress for mortar or cement is less than that for stone. A dam is often built of random rubble or of concrete having a large proportion of "plums" or very large blocks of stone. This "cyclopean" concrete is economical where large blocks of stone can be readily obtained. If built of stones in courses a dam offers less resistance to horizontal shearing and this method is not often employed. The facework may be of dressed stone. The change from this class of work to that adopted for the interior should not be made too abruptly.

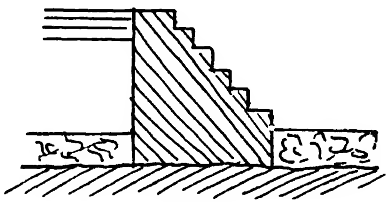


FIG. 167

The ends of the dam are built into the sides of the gorge or valley. Hence a portion of the dam cannot be overturned or pushed forward without being torn away from the side portions. In a long dam the foundations are not at the same level throughout and therefore any two adjacent portions overturned would revolve about axes at different levels and would have to be torn apart.

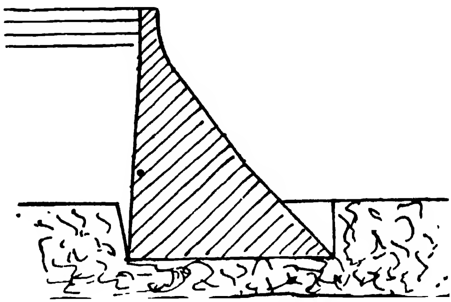
The masonry of a dam must, as already remarked, be of excellent quality. This applies more especially to the upstream facework. If any horizontal crack occurs, water under great pressure will enter it. The crack may possibly extend and fracture take place.

A "high" dam is one in which the compressive stress, due to the weight of the dam, attains the maximum permissible amount. Others are known as "medium" or "low" dams.

It is with the foundations and side connections of a masonry

dam that trouble is most likely to occur. A high dam should not be founded on anything but solid rock from which the fissures have been cut out or plugged by cement grouting. Limestone, sandstone and crystalline rocks are all stronger than masonry and concrete.

With such foundations — the rock being thick — and with a properly designed dam there is no chance of the crushing or rupture of the rock but if the rock is smooth the dam may be pushed along on it. The dam should be in a trench (Fig. 168) or else a series of trenches be cut in the rock and the dam built into them.



A greater danger than the above is that of uplift. If water gets under the dam it tends to lift it and so either to overturn it or to enable it to be pushed downstream. Springs met with in preparing the foundations should be led in pipes to the downstream side of the dam. Most rocks are practically impervious except along seams and these can be dealt with as above. A thin stream of water passing through the dam without much contraction is of little consequence. The pressure head is converted into velocity head.

If instead of solid rock the foundations are thin or broken beds they may be able to support a low or medium dam, but in order to prevent uplift it may be necessary to sink a deep trench (Fig. 169) below the heel of the dam and to fill it with concrete. The figure shows the Howden dam of the Derwent

Valley Waterworks. The base of the dam is 70 feet below the ground. The bottom of the trench is 55 feet lower.

The spillway is over the top of the dam. With the reservoir full and 5 feet of water over the crest, the calculated pressure on the heel of the dam was 4.2 tons and on the toe 7.43 tons

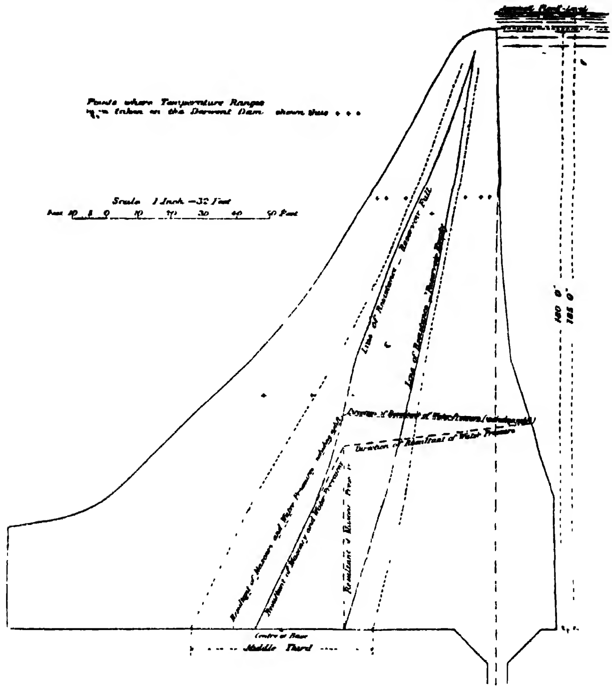


Fig. 169.

per square foot. If it is assumed that water penetrates to the concrete trench, the above figures become 2.45 tons and 8.25 tons respectively. With the reservoir empty they become 9.23 tons and 1.76 tons. The Derwent dam is similar to the Howden dam except that the trench is 38 feet below the base of the dam.

It was found that the sides of the valley were also of broken rock and not water-tight. In order to prevent the escape of water, long wing walls — the lengths were 2,860 feet and 2,600 feet — were run out at right angles to the Howden dam and carried in trenches along the sides of the valley. In the case of the Derwent dam the dip of the strata rendered the above procedure unsuitable and the wing walls were made in the line of the dam and carried into the hill side to lengths of 605 and 792 feet respectively. The tops of all the wing walls were at high flood level, the foundations were carried down to impermeable strata.

Low dams can be built on clay or on layers of rock alternating with soft material. In all cases except hard and thick rock the foundations can be tested by laying down cubes of rock and weighting them. Roughly speaking, pressures on foundations may be 3 or 4 tons per square foot in the case of clay, 4 to 8 tons for shale, 10 to 15 tons for sedimentary rocks, 30 or 40 tons for crystalline rocks but the whole question of the foundations of a masonry dam requires judgment and experience.

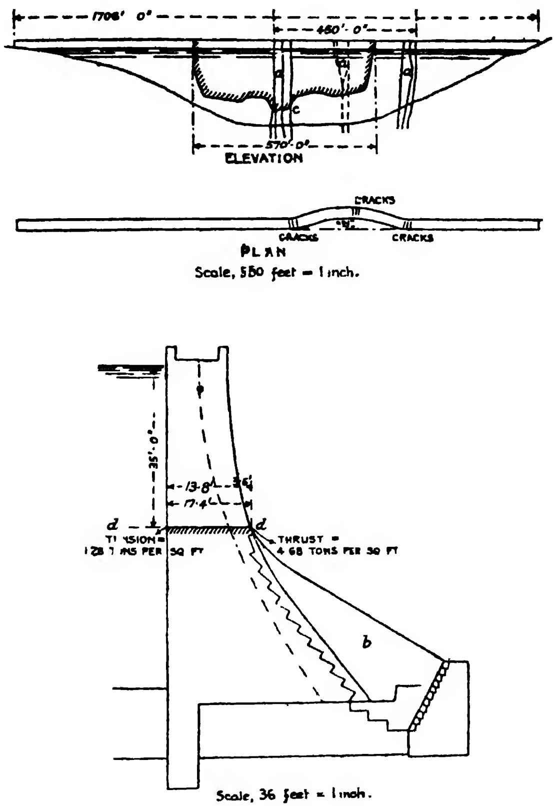
Changes in Temperature. — Owing to contraction during cold weather, cracks frequently appear in masonry dams. The cracks are more or less vertical and normal to the length of the dam. If the dams could all be built during the coldest time of year it would a great advantage. The cracks do not endanger the dam. If the dam has been properly designed each section between two adjacent cracks is safe in itself. It has however lost — to some extent — the support of the adjoining sections. Sometimes a dam is built in sections with expansion joints. First a number of alternate sections are built. Afterwards the other sections are added. It is best that the latter should be far shorter than those of the first set. They can be added during cold weather, but changes of temperature in the interior of heavy dams occur slowly and as much time as possible should be allowed for the first set of sections to become cold. The ends of the sections — they can be in horizontal zig-zags so that streams of water cannot readily pass through — are oiled or painted so that they will not adhere to the next section.

The Assuan dam was thickened and raised some years ago in order to increase the capacity of the reservoir. It had been built mostly in the summer. It had developed numerous cracks transversal to its length. The lower central mass of the dam had cooled down and had an annual range of temperature of only about 13° Fah., the outer layers continuing to have a greater range. The new work was expected to crack but not necessarily in the same places as the old work. In order that the whole might form as homogeneous a work as possible it was considered necessary that the new work should, before being joined to the old, attain the same temperature stage as the old. A gap of a few inches was left between the two and was grouted up after two years had — in the case of any section of the dam — elapsed. The widening was on the sloping — downstream — side and the new work was kept in place by means of 1¼-inch steel rods — one for every square metre of surface — embedded in both parts of the structure.

Failures of Dams. — Out of some hundreds of high masonry dams which have been erected, several are known to have failed. Of these, the Puentes dam was partly founded on piles; and in two, the Habra and Bouzey dams, the rule of the middle third was not attended to. Another dam, not so high, the Austin dam, in Texas, U.S.A., failed seven years after construction. It was 65 feet high and founded on limestone, the width of the base being 66 feet. Springs in the bed and sides of the gorge had, during the construction of the dam, given much trouble, and had, after its completion, forced their way through the underlying rock. At the time of failure 11 feet of water was passing over the dam, which sheared in two places, a length of 440 feet of it being pushed forward for 40 or 50 feet without overturning, but subsequently breaking up. The dam was founded in a trench cut in the rock. The rock on the downstream side of the foundation trench appears to have been worn away by the water, so that there was no longer a trench[1]. It seems also probable that water from upstream found its way under the dam and exercised a lifting force on it and so caused it to slide.

[1] *Scientific American*, 28th April 1900

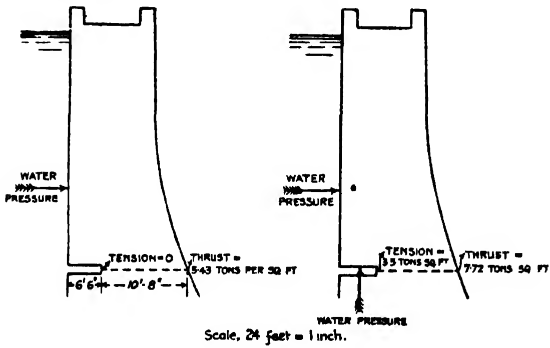
Regarding the Bouzey dam — in France — the rule of the middle third was not attended to. The specific gravity of the masonry was little more than 2. In 1884 a length of



FIGS. 170 and 171.

450 ft. of the dam slipped on the foundation (Fig. 170) the greatest versed sine of the curve being 12 inches. The foundation appears to have been level except for the upstream curtain wall (Fig. 171) which would be sheared through. Vertical

cracks formed at *a a*, at the places where the tension due to bending would be greatest. A strong counterfort *b*, extending over a considerable length of the dam, was then built. In 1895 the dam failed. The line of resistance at *dd* came outside of the middle third and was within 3.5 ft. of the downstream face. A nearly rectangular portion of the dam, 570 feet long, (Fig. 170) was broken away and overturned. The line *dd* shows its lower edge. The fractures had no connection with the earlier cracks except in a small area at *c*. The counterfort *b* was not disturbed. Dr. Unwin calculated that at *dd* there was tension,



FIGS 172 and 173

at the upstream face, of about $1\frac{1}{4}$ tons per square foot and compression at the downstream face of about $4\frac{5}{8}$ tons. A poor cement had been used A horizontal fissure would form at some particular point in the length of the dam, and would become (Fig. 172) about 6.5 feet deep. The tension would then be released and the whole section be in compression, that at the downstream face being 5.5 tons per square foot. If the wall had been a retaining wall for earth it would have stood. But the water pressure in the fissure, acting upwards (Fig. 173) tended to overturn the wall. It would cause tension of 3.5 tons at the upstream, and compression of 7.72 tons at the downstream face. A slice of the dam bounded by vertical cross-sections, would, by itself, have been quickly overturned. But every

such slice was connected with the stable part beyond it. It was not until the fissure had extended and become some 500 feet long, that the overturning moment, due to the horizontal water-pressure and the upward pressure in the fissure, was great enough to overturn the rectangular mass of masonry and also to tear it away at its ends [1].

The horizontal thrust of the water causes a tensile stress on the foundation of a dam on the line M N (Fig. 166). Suppose that the rock has only the thickness M R. There is tension in M N, and possibly compression in N R. It is assumed that, along the base M P, there is perfect union between the dam and the rock. The tension to which the rock is occasionally subjected owing to changes of temperature may exceed any tension due to the water-pressure, but it is possible that the tension occurring from both causes might cause a crack at M N, and that this might extend to R The dam, with the rock M R Q P adhering to it, tends to rotate about the point P. The tendency to rotate will be enhanced if water enters M R, and still more if it enters R Q. No rotation can, however, take place unless the rock is splintered away at M R and fractures at P Q. It has been suggested that the upstream face of the dam be made curved as shown by the dotted line. This would shift the chief tension to *m n*, and the dam, with the rock beneath it and the weight of the water above the curved portion, would obviously offer an increased resistance to rotation about P. The cost of the dam would be increased. The danger of a crack forming at M N seems to exist only when there is a thin upper stratum of rock not firmly connected to rock below. When this condition is believed to exist, a high masonry dam, if built at all, should have the upstream face curved as above described. In the case of any existing dam of great height, when the above condition is suspected to exist, the reservoir might be laid dry, and if any crack at M N is discovered a curved portion could be added; but in this case the union between the new and the old work would be imperfect, and the curve should start from high up on the upstream face of the dam. It has been suggested that asphalt or some

[1] *Proc. Inst C. E.*, Vol. CXXVI p, 95

impervious material be laid on the rock to prevent water from entering any crack. It would, however, not only have to be laid upstream of the dam, but to extend under part of the dam, and thus weaken it to some extent.

Some Details. — The weights of masonry and concrete per cubic foot vary with the constituents used and the proportions in which they are used. Some masonry with close joints may weigh 164 lbs. per cubic foot, concrete 152 to 140 lbs. per cubic foot. The corresponding specific gravities are 2.62, 2.43 and 2.24. The maximum compressive and other stresses to be allowed on the materials used for any masonry dam should be decided for each case in the light of general and local experience. On concrete or ordinary rubble masonry the compressive stress allowed is often 15 tons per square foot but sometimes 20 tons; on masonry of cut stone it may be 30 tons or more.

When ice forms on a reservoir it may contract and crack with a fall in temperature. Ice then forms in the cracks and a rise in temperature causes expansion of the whole. In the case of an earthen dam, owing to the flat side slope, no harm results beyond possible damage to the pitching but thick ice may cause enormous pressure on a masonry dam. The crushing strength of thick ice may be half a ton per square inch. The largest reservoirs with masonry dams, if liable to be frozen over at all, generally have low water in winter. In any case in which thick ice is likely to be formed with a high water level and where an arched dam is impracticable, arrangements should be made for breaking up the ice. At certain seasons drift ice is afloat and this, under the influence of wind, may cause considerable stress on a dam. This can be allowed for in the design — if necessary — or special structures be arranged to protect the dam.

In the case of a masonry dam the questions of the outlet and waste weir are less troublesome than with an earthen dam. It is best to have both away from the dam but in a masonry dam the outlet or outlets can be in the dam itself without any complication, and the dam is generally long enough to admit of the waste weir being formed by utilising a portion

of the crest. The crest can be rounded in such a way that the stream of water will not spring clear of it and so perhaps form a partial vacuum and increase the pressure in a downstream direction. The downstream face of the dam near the toe is curved and brought to a very flat slope so that the water shall not damage or wear away the rock.

While a masonry dam is under construction the water is often diverted by means of a tunnel. In building a dam the top should be kept irregular. If a large portion is all built up to one level there is, as it were, a large horizontal joint which offers reduced resistance to shearing.

Arched Dams. — A masonry dam, instead of being straight, can be curved so that the water pressure causes it to act as an arch. The sides of the valley or gorge must be of rock capable of withstanding the thrust of the arch, and be cut to the proper angle to receive it.

In an arch which holds up water, the pressure is everywhere normal to the extrados of the arch The line of pressure within the arch follows the curve of the arch and every part of the arch must be in compression. If S is the admissible pressure on a voussoir, P the water pressure at any depth below the high flood level — both in tons per square foot — and R the radius of the arch in feet, then the required thickness of the arch at that depth is approximately, $T = \frac{PR}{S}$.

The thickness increases with the depth The thickness at the top, theoretically zero, must of course in practice be appreciable. The section of the dam in practice is wedge-shaped, both faces having batters.

Let it be supposed that the bottom of the gorge is horizontal and its sides vertical. The base of the dam being firmly connected to the rock below it, the lowest part of the dam cannot yield to the water pressure and cannot act as an arch. The greater the height above the base the more the dam can yield and the more the structure acts as an arch. At a considerable height it acts almost entirely as an arch and, if R is not great, its thickness can be much less than if it were a straight or "gravity" dam. There can be no tension on its

upstream face. Suppose the height of the dam to be 216 feet. At the base, P is 6 tons per square foot. If R is 100 feet and S is 15 tons per square foot, T is 40 feet. The thickness at the top may be 10 feet.

R must always be small. If great, T comes out so large that nothing is gained by adopting the arched type of dam. The most economical angle for the arch is 120° to 135°. Thus the span cannot be great. The gorge must be narrow.

A structure built as an arch, but not under pressure, cannot act as an arch without slight movement in it taking place. It might appear possible that movement could take place above a certain level while there was no movement below it, and that shearing might thus take place — either along a portion of the length of the dam or along the whole of it — and that water might thus find an entry and exercise its full pressure. It will probably, however, be found on calculation that the section is sufficient to resist the shearing stress. Even if shear does occur the upper part of the sheared length cannot move appreciably. Neither overturning nor sliding can occur without the crushing of the material of the dam or of the abutment. The lower part of the dam is equally safe. Even if, for some reason, water had found an entry below it as well as above it, no sliding could take place.

The following are the dimensions of some arched dams.

Name	Radius at base (Feet)	Depth of base below H F L (Feet)	Thickness at base (Feet)	Thickness at top (Feet)	Batter	
					Up-stream	Down-stream
Bear Valley	335	48	8.42	3.2	1 in 8	nil
Pathfinder	186	210	94	10	1 in 4	3 in 20
Shoshone	153	245	60	10	1 in 4	3 in 20

The Shoshone dam is shown in Fig. 174. In all cases the base thickness is much less than it would be in the case of a gravity dam. The Bear Valley dam is remarkable. The pressure at 48 feet depth, calculated from the arch formula is about 60 tons per square foot. At this depth — 16 feet above the base — the thickness, in going downwards, abruptly

increases. This arrangement does not appear to be correct since strains are set up at sharp angles. The dam stood for many years. It has since been replaced by a higher one of a different type.

If, as is common, the sides of the gorge are sloping so that the span of the arch is small at its lowest part, the amount of yielding in order that it may act as an arch at a low level, is very small, and the greater part of the dam acts as an arch. In such a gorge it is convenient to reduce the radius of the arch at the base of the dam — letting it increase as the width increases — and so reduce the thickness.

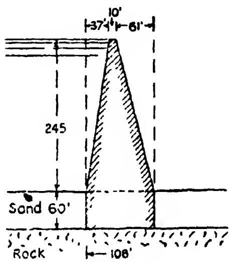


FIG. 174

Multiple Arch Dams. — If a bridge consisting of piers and arches — without any superstructure — is laid on its side in a stream, with the arches upstream, it forms a multiple arch dam, the piers forming the buttresses. Sometimes the arches are semicircular and the whole dam is curved (Fig. 175). Generally the arches are segmental and they are neither horizontal as in a bridge nor vertical as in the case just mentioned, but are inclined at about 45° the buttresses being triangular (Fig. 176). By this arrangement a large spread for the foundations is secured. Multiple arch dams can thus be used in streams of considerable width and with beds of clay, shale or soft rock. They are perhaps on the whole more used than single arch dams. When low they are more economical than any solid dams.

In the case of multiple arch dams and of the slab dams described below, a continuous floor covering the whole

structure is necessary except in the case of rock. A cistern for the falling water can be made as for a weir, or by means of a wall built on the floor, but this, if the floor is at the bed

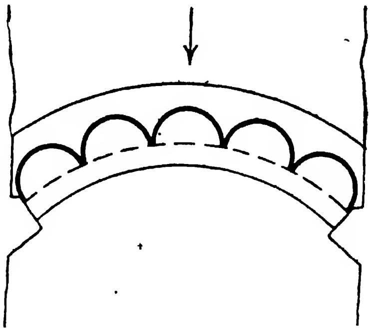


FIG 175

level of the stream, gives rise to a fresh fall which must be suitably dealt with. If the bed and sides are not of rock scour must be provided against as in the case of a weir.

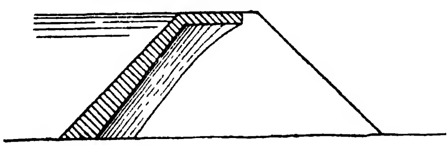


FIG. 176

Miscellaneous Dams Slab dams of reinforced concrete consist of an inclined deck resting on buttresses. They are inferior to multiple arched dams because of the limitation of the span and are used on a smaller scale. They are then more economical than solid dams. They may be of various forms, see for instance Fig. 144, p. 275.

A "hydraulic-fill" dam is one in which jets of water under high pressure are used to loosen and dissolve the earth — this is termed "sluicing" — and the water then carries away the material through pipes or flumes and deposits it on the site of the dam. Banks forming the upstream and downstream

portions of the dam are previously constructed, so as to form a tank for the liquid material. The tank is filled to a moderate depth. The solid parts settle and the liquid is drained off. The settled material becomes compact and fairly dry. There may be much difficulty in effecting this if the material is clay. It must, however, be effected before more liquid is added, otherwise—especially in the case of clay—the whole may become semi-liquid, exercise great pressure on the banks and perhaps burst through them.

If the earth contains materials of various degrees of coarseness, they can to a great extent be distributed to the positions in which they are required. When the tank has been filled nearly to the level of the top of the bank the latter can be raised. If water under pressure for the sluicing cannot otherwise be obtained pumping can be resorted to.

A "rock-fill" dam consists chiefly of loose stone and rock which is thrown into the gorge. It is used where the bed is of rock or is at least very hard. The upstream slope may be 1 to 1 and the downstream slope 2 to 1. Such dams have been made to heights of 150 ft., the top width being 12 ft. The largest stones or rocks should be placed along the downstream toe. The dam is made water-tight by a "deck" of steel plates or creosoted timber, laid over the upstream slope. The deck runs into the rock at the bottom and sides of the gorge and is attached to timbers which run into the dam. Sometimes instead of a deck there is a vertical core-sheet of steel plates resting on a concrete base. The stone at the faces of the dam—and along both faces of the core if any—is hand-packed. When steel plating is used it is provided with expansion joints and is coated with asphalt. Its thickness may be $\frac{3}{8}$ inch at the bottom and $\frac{1}{4}$ inch at the top.

In a few cases in America the upstream part of a dam has been made of earth and the downstream part of rock fill. In such a case the stone should be graded, the finest grades being used next the earth and the coarseness reduced gradually.

Note on Arched Dams (p. 330). A more accurate formula for the thickness of the arch is $T = R\left(1 - \sqrt{1 - \frac{2P}{S}}\right)$ where R is the radius of the extrados.

CHAPTER X.

Works on Some Great Rivers.

[For preliminary information see pp. 64–69 and pp. 285–6.]

Art. 1. Bridges and Weirs. In the railway bridges across great shifting Indian rivers it was once usual to make the total span too great. Now it is the rule to make the span far less than the width of the channel. The stream during floods scours out a deep channel through the bridge with great rapidity, and no heading up worth mentioning occurs. The foundations of the piers are very deep, frequently 50 ft. below the lowest point of the river bed which can be found anywhere within several miles of the bridge. The total span of the bridge can be arrived at by considering a general cross-section of the river as it is in high flood and assuming that scour to the depth of the lowest point, found as above, will take place in one-third of the span of the bridge. The span can then be so fixed as to give practically no heading up. It is not assumed that there will be no increase in velocity through the bridge. The velocity in the deep scoured portions will be increased.

The piers are protected by loose stone (Fig. 177). The spans vary from 100 to 250 ft. The bridge over the river Chenab at Wazirabad had originally (1876) sixty-four spans of 145 ft. each. After many years the number of spans was reduced to twenty-eight and subsequently to seventeen. With a

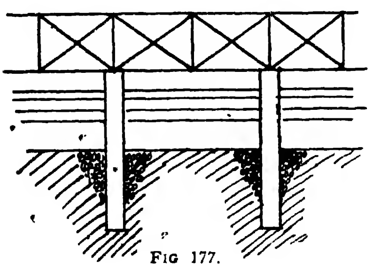


Fig 177.

very long bridge, the current of the shifting stream is more likely to strike the bridge obliquely, though this is not the chief reason for reducing the length. Between

piers long spans, say 250 ft., have been found to be better than shorter spans; the cost of the stone protection round the piers is, of course, less.[1]

To guide the river into the bridge it was once usual to construct systems of spurs. Now Bell's guide banks (Fig. 178) are always used. They are discussed in the paper by Spring. They are pitched with loose stone; the spaces

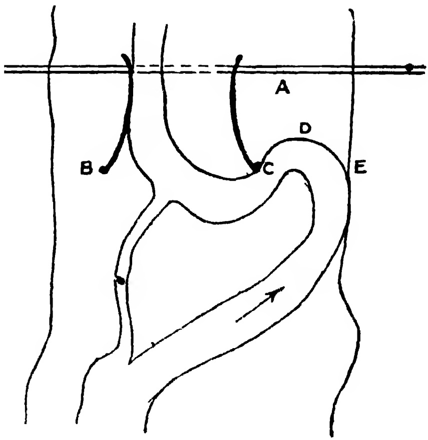


FIG 178.

behind them become filled with water, at least during floods, and are meant to be silted up. An opening in the railway embankment should be provided at A, and another on the opposite side of the river, to ensure a constant flow of water to provide silt; they should not, of course, be too large.

The chief danger to which a guide bank is liable is outflanking when the stream assumes the position shown. To guard against this danger it is necessary to have very strong and massive "heads" of stone at B and C. When the bank CDE becomes a semicircle or thereabouts, the stream takes a short cut—during a flood—across the sand-

[1] *Government of India Technical Paper*, No. 153, "River Training and Control on the Guide Bank System" (Spring).

bank, and to encourage this an artificial cut can be dug at the season of low water on any suitable line. It should be kept closed till the water in it will be fairly deep. (See p. 105.)

If the guide banks were made with an increased width of opening at the upper end, this would reduce the chance of outflanking, but would increase the danger from a direct attack, such as indicated in the figure, on the left bank. It has been suggested that the width BC should be less than at the bridge, but this seems most undesirable. Probably the form shown is the proper one. The length of the guide bank upstream of the bridge is made about equal to the span of the bridge. If made less than this, the river CDE might cut into the line of railway. The length of guide bank downstream of the bridge is generally 300 to 500 ft., being greater as the velocity of the river is greater and the sand of its bed finer.

The pitching of the guide bank (Fig. 179) has a slope of 2 to 1, and consists of quarried blocks of stone loosely laid,

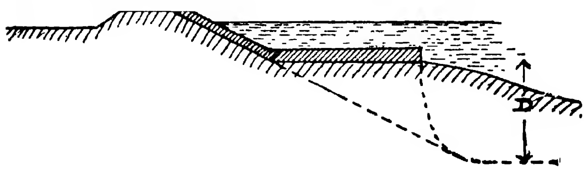


FIG 179.

the largest blocks weighing perhaps 120 lbs. The apron—more or less horizontal—is laid at the time of low water on the sandbank or bed of the stream. If necessary the ground is specially levelled for it. When scour occurs it sinks till it assumes, more or less, the position shown by the sloping dotted line. In the case of excessive scour it may slip down farther. The pitching on the upper slope may follow or it may possibly be damaged in some way. Along the top of the bank there is generally a line of rails so that stone from reserve stacks, which are placed at intervals along the bank, can be quickly brought to the spot and placed on the slope.

The following dimensions of the apron are given by Spring. The probable maximum depth of scour can be calculated as explained above. If this depth, measured from the toe of the slope pitching, is D, and if T is the thickness considered necessary for the slope pitching, then the width of the apron should be 1.5 D, and its thickness 1.25 T next the slope and 2.8 T next the river. It will then be able to cover the scoured slope to a thickness of 1.25 T. This thickness is made greater than T because the stone is not likely to slip quite regularly. The thickness T should, according to Spring, be 16 inches to 52 inches, being least with a slow current and a channel of coarse sand, and greatest with a more rapid current and fine sand; but since the sand is generally finer as the current is slower, it would appear that a thickness of about 3 ft. would generally be suitable. Under the rough stone there should be smaller pieces or bricks.

The idea—once more or less current—that the permanent training works of a structure like a railway bridge fix the position of the river for quite a long distance upstream is altogether erroneous. The stream, as has been seen, may approach the works from almost any direction. When the irrigation engineers decided recently to make a canal headworks in the Sutlej near Ferozepore they selected a site 700 ft. downstream of the railway bridge.

In the above and three other weirs recently built on the Sutlej for the headworks of irrigation canals, the cross section is on the whole somewhat similar to that of the Merala weir (Fig. 124), but instead of the three lines of deep wells there is one line of sheet piling. The lengths of the weirs are smaller compared to the width of the river, so that the discharge per foot run of weir in floods is rather heavier.

In the above weirs the width BC (Fig. 178) has been made equal to or less than the length of the weir. The reasons for this are not at all clear. The form shown in Fig. 178 seems to be correct and any reduction in the width BC to be unsound. In the above cases the upstream guide-

banks are long. The object is said to be to obtain a straight flow to the weir, but the lengths are excessive. Those of the downstream banks—they are straight and at right-angles to the weir—are in one case no less than 800 ft. The object of this is wholly obscure. The banks form a confined channel tending to cause scour unless the whole bed between them is pitched. The design of guide banks at canal head works has little or nothing to do with "retired embankments," which are usually miles apart and are useful only in floods. They prevent the spill water from breaking into the canals and compel it to go over the weir. See also Appendix A.

Art. 2. Works on the Indus. Of all shifting rivers those of the Punjab shift the most. The Indus is larger and more shifting than any. The other five unite and join it before it leaves the Punjab and enters Sind. The discharge of the Indus above the junction is, in the winter, about 22,000 c. ft. per second. In a high flood it is perhaps thirty times as great.

Westward tendency near Dera Ghazi Khan. In a reach some 70 miles in length near the south of the Punjab, the Indus in its changes attacks the right bank more than the left, and thus the result, at the end of a series of years, is on the whole a westward movement of the main stream and of the channel. The country on both banks is mostly below flood level, and there are on both banks long flood embankments. In the thirty-five years from 1875 to 1910 the westward movement amounted in places to 2 or 3 miles. It not only threatened to destroy the town but it often destroyed a flood embankment, which had then to be reconstructed farther away from the river and in lower ground, the cost being thus increased and also the danger of breaches. The amount of westerly movement is found by comparing the centre line of the main stream with its position in a former year. The two lines cross each other here and there, the river having moved east in some places and west in others.

The causes of the westward movement are not known.

Fig. 180 shows that it is not due to any general curvature of the course of the river.

The eastern bank of the river is in many places fringed with forests, consisting mostly of not very large trees. Their roots offer some resistance to erosion. On the Ravi just above the head of the Sidhnai Canal there is a perfectly straight and permanent reach about 6 miles long. No such permanent reach exists on any other such river.

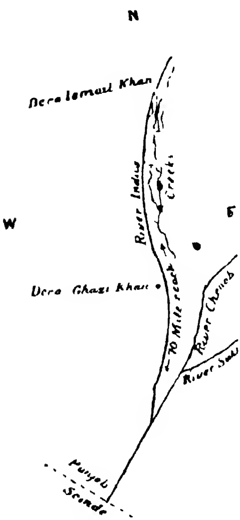


FIG 180.

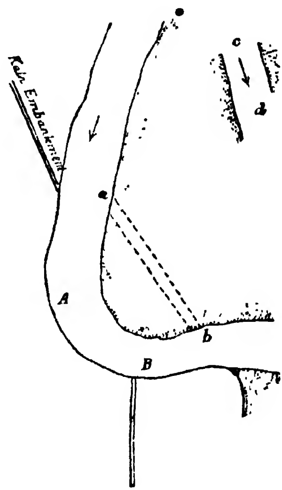


FIG. 181.

Trees exist along both edges—there is a tradition that they were planted—and their roots prevent erosion. The forests on the east of the Indus must tend to prevent the stream from moving east whenever it swings to that side. This may possibly be one cause of the westerly tendency.

Another cause may be the rotation of the earth. Taking the average southerly velocity of the stream during floods—that is when the changes occur—to be 6 ft. per second, the

distance traversed in an hour is 3.6 miles, which represents, in the latitude in question, a difference in velocity about the axis of the earth of .66 ft. per second. This difference takes place in an hour and it may seem to be small, but the body of water affected is immense. In the 70-mile reach the main stream is frequently tending to take a new course and the change may or may not occur before the flood subsides. In some cases the westward pressure may have a deciding influence.

In other reaches or in other rivers flowing southwards there may be no westerly tendency, but the volumes or velocities may be less and there may be nullifying causes such as forests, high banks (p. 56) or hardness of soil.

The whole river—in the Punjab—is said to have once flowed many miles to the east of its present course and then to have moved slowly westward. About the year 1400 A.D. the river, both in the 70-mile reach and for some miles north and south of it, undoubtedly flowed some 6 or 8 miles to the west of its present course. Its western bank is still traceable. When Dera Ghazi Khan was founded, 400 years ago, the river was moving eastward. The recent and present—for it still continues—westward tendency seems to be one of a series of great and slow changes whose cause is doubtful. The " draw " of the western canals is negligible and that of the eastern canals is as great.

Spurs (Chap. IV, Art. 3). Spurs have been made—in the low-water season—in two places where the Indus was eroding its right bank upstream of Dera Ghazi Khan. The idea was to stop the westerly movement near the town. If the bank could be protected for a time the river would in due course leave it and move away. The spurs were made of small trees dragged to the river's edge by bullocks and weighted with nets filled with boulders brought on camels from the Suliman hills, twenty miles to the west. The trees sank and others were piled on them. In the bend AB (Fig. 181) 17 miles north of the town, ten spurs, extending over half a mile of bank, were made in 1878–79. The lengths attained were generally 20 to 30 ft., but one longer

one was easily made in a shallow place where the current had deserted the bank. When the river rose in 1879 it took a new course *cd*, which began five miles upstream of the spurs. Probably this change was not due at all to the spurs. The old channel soon silted up and the spurs became buried in silt. In 1908 they were in the midst of fields. By 1923 the river had again cut in almost to the 1878 position. In a reach nearer the town fifteen spurs were made in 1882–83. When the river rose in June, 1883, it carried away eight spurs in a few weeks. At the remaining spurs the river abandoned any westward tendency and they became silted up. They disappeared in the years 1885, 1886 and 1887, when the river again attacked the bank. Since bank erosion almost ceases at low water, the spurs did not do much good.

Floating Spurs. In connection with the spurs of 1882–83 use was made of barges carrying shutters which could be let down like drop-keels. Several barges in a row gave a long line of shutters forming a floating spur. The principle was sound in one respect; the spur could not be destroyed by the river. But the shutters reached down only 5 ft. below the water; there must have been a rush of water under them, analogous to the rush round the end of a spur, and tending to cause scour of the bed. The shutter barges were

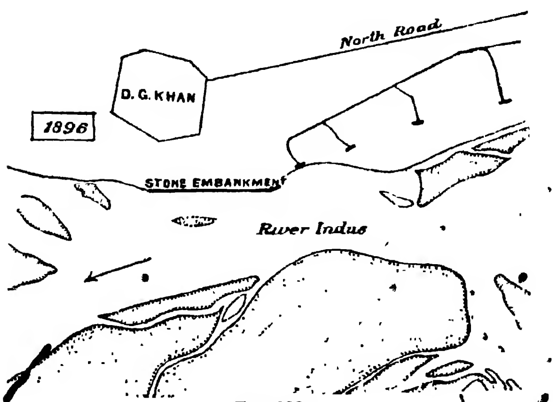


FIG. 182.

soon given up. They might be more effective if the shutters could be made to reach down to the bed.

The Stone Embankment. In 1888–89 (Figs. 182 and 183) an embankment a mile long was constructed on Bell's

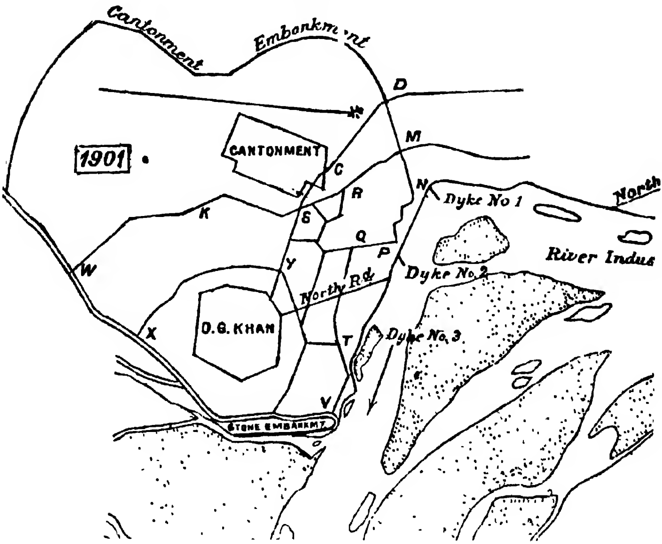


FIG 183

principle to protect Dera Ghazi Khan. The town contained some 20,000 people. It was owned by private individuals, but its destruction might have been succeeded by that of the "cantonment" (shown only in Fig. 183). Up till 1895 the main stream did not touch the work. In 1895 it cut in above the work and the nose of the latter was strengthened. In 1896 four earthen spurs with cross heads of stone were made. From 1896 to 1898 the main stream was in contact with the work, but did little damage. In 1900 it cut in deeply and carried away the four spurs and their heads. The upper end of the work was strengthened. No money for heavier work was forthcoming. In 1901 this difficulty was overcome, the Viceroy of India, Lord Curzon, having visited the town.

In 1902 an attempt was made to obstruct the western stream—it was the main stream—by means of hurdle dykes (Fig. 183), each dyke consisting of three lines of very long piles driven into the bed of the stream—which was to be protected with mattresses made of fascines—and extending right across it with their heads above flood level. The idea was to cause silt to deposit and the channel to become thus choked up. The work was thus partly a diversion work, and not merely a protective work like the others. It was in progress in May, 1902, when an unusually early flood put a stop to it. The dykes had advanced some way from the right bank of the stream, but none had been completed. Dykes 1 and 2 were for the most part carried away. The river took a new course, starting from a point far upstream, the western channel became a creek, and the remains of the dykes were soon embedded in silt.

In June, 1908, the main stream was again as in Fig. 183, but farther east, its right bank pointing to the site of dyke No. 3, and then turning to the east. The nose of the embankment was strengthened and the stream behind it became semi-circular, but it did not take a short cut. It eroded the rear of the embankment, the stone fell in and was swallowed up and the stream broke through the embankment, and before long destroyed it and part of the town, leaving the nose standing for a time as an island. By 1910 most of the town had gone.

The embankment should have been made in a curve, starting from its southern end and swinging inland, with its centre at the middle of the town. Some gardens or buildings would have come in the way and some others would have been left unprotected, but such considerations should have had little weight. In 1901, instead of a new kind of work being tried—the efficiency of stone embankments had been amply tested at the railway bridges—the embankment might have been extended so far as funds permitted.

Leading Cuts. Attempts to train the Indus by excavating cuts to lead the river to the east have occasionally been made, but have not succeeded. Cuts made by manual

labour can only be dug to a foot or two below the water level because of the sub-soil water.

Groynes. A groyne is the name locally given to an earthen bank thrown across a sand-bank. It generally crosses a creek. While it lasts it prevents the creek from flowing and becoming larger. It strongly tends to increase the silt deposit on the sand-bank upstream of it and so to add material to the west bank of the river. On its downstream side it perhaps does not increase the deposit which the sand-bank would have received without the groyne. It may last for years or it may be soon destroyed by erosion or by breaching. Its length may in some cases be a mile or more. It is not usually made unless the creek or creeks to be crossed are small or go dry in winter. The closure of a creek with a heavy discharge is expensive. Where the main stream is near the west bank no groyne can be made. Groynes have been regularly made in the greater part of the 70-mile reach ever since 1889. Up to 1908 about 28 had been made.

The top of a groyne is generally about 7 ft. wide and 3 ft. above high flood level. The material is usually sandy and may be pure sand. The sides slope at about 2 to 1 and are protected by light fascines made of tamarisk. The landward end abuts on ground above ordinary flood level or is carried on till it meets a flood embankment. The riverward end is generally at a point where the sand-bank is high. To prevent damage by a rush of water along the groyne, and especially round its end, short spurs can be made along it at intervals, and a strong protection of brushwood and stakes at its end. From May to September each groyne has to be closely watched, any damage made good and leakages stopped. If a very high flood occurs, it may be necessary to cut the groynes and let them be destroyed in order to save the flood embankments. These generally run parallel to the general direction of the river. Near the town there are several local embankments.

As the river has moved west it has left some permanent creeks on the east (Fig. 180). They are in flow in the flood

season and the people throw dams across them for irrigation purposes. An idea that this is a cause of the western movement of the river is erroneous. The creeks are too long and too much silted at their heads to enlarge themselves if left alone. In forty years the closing of creeks on the west has not even stopped the movement to the west.

It is probable that, but for the groynes, the river would have moved farther west than it has moved. The closures of creeks also increase the supplies in the irrigation canals. These are known as "Inundation canals" because they flow only when the river is high.

Art 3. Avulsions and the Sukkur Barrage. The rarity of avulsions in rivers of the Punjab type has been explained in Chap. III (p. 67). In the Punjab there is no record of the occurrence of any avulsion.

On leaving the Punjab the Indus enters Sind. Its character becomes intermediate between that of the Punjab rivers and of the Mississippi. The width of the river is less than in the Punjab. The channel can become crooked, but does not often form pronounced loops. The only avulsion of which there is any record is that which occurred sixteen miles below Sukkur (Fig. 184) in 1906–07. The river shifts its course in the usual manner (see the dotted lines just above Sukkur) by scouring at one side and silting at the other.

The river in Sind is in alluvial soil except in the Sukkur gorge, which is rock. In whatever manner the stream shifts above the gorge, the lower end of the reach remains in the gorge. It has a general tendency—but less than in the reach considered in Art. 2—to move westwards and this has left it crooked near the gorge. This does not imply special danger at Sukkur.

The usual explanation of the formation of bends is that some obstruction, say on the left, causes some bank erosion on the right. Thus a slight bend is formed. The water leaving this bend cuts in on the left lower down. In this way a succession of bends is started and each bend tends

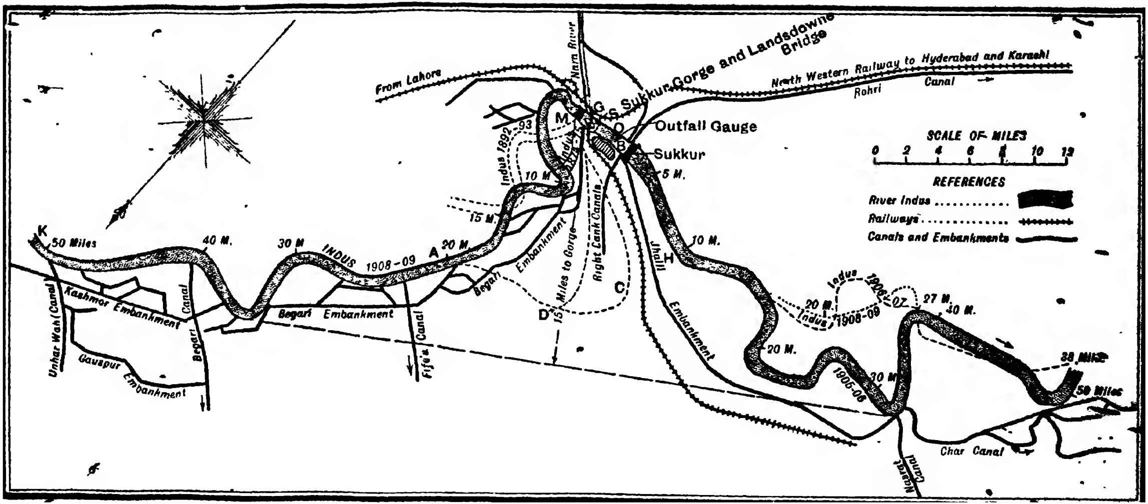


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FIG 184.

to increase; especially if, as is common, the bed is rising and the slope increasing. A similar set of bends is formed in other reaches. In all of them the velocity in time becomes reduced. If any one bend increases much, the velocity there is checked and its action ceases. In any straight reaches it remains high and there is every chance of bends starting there also. The whole tendency is for small swings to either side.

Thus if in any river we consider a 50-mile reach in which the general conditions are fairly uniform, it will not be found that one-half of the reach—say the lower one—contains all the bends and the other none. The causes producing the bends are the same in both halves. The bends will in the end tend to become equal in the two halves. Similarly, the causes producing bends operate in both directions. The river swings this way and that. It is rare for it to lengthen itself by one very large bend, taking it far out of its general course. The plan shows that in the past the river above Sukkur has shifted this way and that. There is no evidence that it has ever adopted any course approximating to that marked ACM. It is most improbable that it will adopt such a course. It would involve a great excess of bends in the lower half of the reach KM and a great deviation at C from the general line KM. Such a course might be taken if the point K moved so far west as to bring D into the general line. This is a very remote contingency.

Moreover, an avulsion occurs only during a high flood. It can be absolutely prevented by a simple embankment.

The Begari and other marginal embankments are for the purpose of preventing floods from spreading over the country. They have nothing to do with the ordinary shifting of the river. The river frequently shifts so as to threaten to cut into an embankment. In that case another is made in its rear. This is constantly done, both in the Punjab and in Sind. The plan shows many embankments. But until an embankment is cut into or breached in a flood an avulsion cannot occur. The avulsion of 1906,

below Sukkur could if necessary, and if foreseen in time, have been prevented by embankments. Breaches in the Indus embankments sometimes occur. They are generally closed before long. They never cause avulsions. Of course, there would be an embankment on some such line as CM. As long as it stood it would prevent an avulsion. Breaches in it would be specially guarded against and could be prevented. There would also be special arrangements for closing any breach if it did occur. The danger of avulsion as matters now stand is well-nigh negligible.

If ever the stream got to a line such as ADM it would meet with the rock which extends from the Sukkur gorge north-westward for three or four miles. Thence to D a stone-protected guide bank could be made, though it might be deferred owing to the expense and the river might cease to move nearer to H.

The general public, knowing that avulsions sometimes occur, but knowing no more, is often in fear of them. It was sometimes so during high floods at Dera Ghazi Khan in the Punjab, where no avulsions ever occur. The public draw no distinction between the usual flood which merely inundates and the almost unknown one which sometimes scours a new channel across country.

As regards Sukkur, even professional engineers have expressed fears, based on the idea that the barrage now being built at Sukkur may, by heading up the water, increase the chance of an avulsion. Probably they failed to recognize the negligible character of the danger as matters now are.

The gorge GS is narrow, very deep and of rock. From G to B it widens steadily and becomes a mile wide, the bank being high and of hard clay and, therefore, permanent.

In high floods the water has a fall of 2 or 3 ft. in GS, but at lower stages the fall quickly becomes negligible. A project in which the barrage was to have been just above the gorge was abandoned. Very expensive guide banks would have been necessary in order to cause the river to enter the gorge in a suitable direction. In the sanctioned scheme the barrage is at B. This site has the enormous advantage of

requiring no guide banks. The heading up at the barrage is not required during high floods. As regards water-level the site is practically as good as the other.

The barrage just above the gorge was designed to hold up the water in the lower stages of the river to 192.0 ft. above sea-level. The flood level here is 202.3. The increased danger of avulsion would have been nil. In the new project the canal bed levels are placed higher than before. This reduces the cost of the canals, but requires more heading up in the river.

The barrage is designed to hold up the water to 194.5, which is very slightly above the average flood level of July. In June and September an average heading up of about 1.5 ft. will be necessary, in May of 4.5 ft., in winter of 8 ft. These "affluxes," of course, get less in going upstream and vanish at points far upstream. They merely prolong the period of moderate flood level and to a slight extent the period of careful watching of the flood embankments. This can be faced. Thus, under the existing scheme, the direct effect of the heading up will be to increase by an appreciable, but very small, amount a danger which is at present almost negligible.

There remains the question of indirect danger owing to silt deposit. When the river is headed up 8 ft. at the barrage the backwater will extend for some twenty miles upstream. There will be some silt deposit in the river channel. It is argued that the deposits may still be there when the high floods come, and that the height of the floods may thus be increased. The quantity of silt carried by the Indus at the outfall site has been carefully observed for many years. The quantity carried past Sukkur in a year may be as much as 10,000 million cubic feet, which, if deposited, would be equivalent to a depth of some 20 ft. over the whole river channel for 20 miles. The surface slopes, when there is heading up, are shown in Fig. 185 by dotted lines. In the six winter months the slope is very flat, and V is about half the normal figure, but there is then very little silt. In May and October there is rather

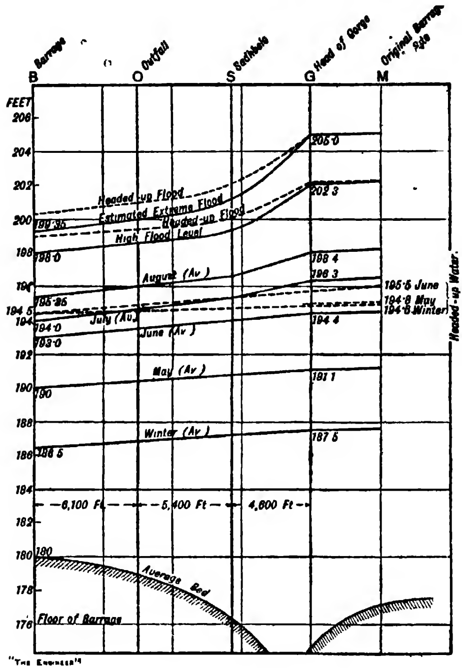


FIG. 185.

more silt, but V is higher. The sand rolled along the bed is probably much less than that carried. In winter there is little. At other times it will pass under the gates, which will be partly raised. The reductions in V, however, reduce the rolling power of the stream as well as its carrying power. The total quantity of silt to be dealt with is thus somewhat greater than that stated above. In the whole of the eight months, October to May, the silt deposit may be about 1.5 ft. over the 20 miles of river, say 3 ft. at the barrage and zero at the upper end.

In June and September heading up—with V reduced from normal by some 22 per cent.—will alternate with moderate floods in which the gates will be raised. In

July and August the floods will generally pass freely. It is in these four flood months that the vast bulk of the silt is transported.

General experience enables the action of the river to be foretold. If a given volume of water is available for cleaning out a drain, the best way to utilize it is to send it down in flushes. In the case of a regulator in a canal, it is found that silt which is deposited in the upstream reach, owing to the holding up of the water, is generally washed out when the holding up ceases (p. 71). The greatest depth of silt is at the regulator, and a sudden drop in the bed occurs there. That eminently favours quick removal by a rush of water. The supply being steady, the holding up should perhaps extend over not more than half the total time. In like manner, the Indus will free itself from the silt deposited by the holdings up. These will extend over more than half the time, but not over a period covering half the total discharge.

Only the early floods—those in May or June—will be made higher by the deposits. After them will come the prodigious power of the big floods. The rushes of water which are required will be there. The project engineers and others acquainted with the Indus have seen it in flood. They have seen how quickly it can remove such deposits as already occur above the gorge. There is no fear that its bed will be permanently raised.

CHAPTER XI.

Tidal Waters and Works.

Art. 1. Tides and Tidal Rivers. The tide which flows in from the outer sea is called the "flood tide." It meets with obstruction from shallow water or from the coast. This causes the rise of the tide. The conditions vary enormously, and with them the rise. Most of the rise is due to momentum. It is greatest when the flow is up a funnel-shaped estuary. In all cases friction causes the energy of the "ebb tide" to be less than that of the flood tide.

The period between one tide and the next, e.g., from high water to high water, is about 12 hours, 25 minutes. At a "spring" tide the rise and fall of the tide are greater than usual; at a "neap" tide less. The times and levels of high and low water at various places have been ascertained and are recorded. The levels are liable to be affected by winds, but not usually in any such simple manner as, for instance, that an on-shore wind raises the level. When a river or estuary is shallow and the range of the tide is great, so that its rise is rapid, the flood tide in some cases advances in the form of a wave or "bore"; that of the Severn is well known. The rise and fall of the tide are least rapid near the turns of the tide; if the time from the beginning to the end of the flow is divided into six equal parts, the proportional rise of the water is approximately as follows. And similarly with the fall during the ebb.

Time . . .	1	2	3	4	5	6
Rise of water .	.067	.25	.5	.75	.933	1

Tidal waters flowing up and down the lower portions of rivers render them to an enormous degree more suitable for navigation and, especially if they become enlarged and

form estuaries, more capable of being altered by training works. The waters are frequently charged, more or less, with silt.

Let AB (Fig. 186) be the surface of the lower part of a river, supposed to be of uniform width, and let B be the mean sea-level. As the tide rises to D or falls to F, the river water is headed up to AD or drawn down to AF. If the rise of the tide BH is such that the river discharge cannot keep pace with it so as to fill up the whole space between A and H, there is a flow of sea water from H to some point M lower than A and H, and of river water from A to M. If the tide now turns and the water-level H begins to fall, there is still flow along HM. For a brief period it is due to momentum, but it continues until, by the rise of the water-level at M and the fall at H, the surface has assumed the form indicated by the dotted line ANJ. While this is happening, the point corresponding to M—where the concave curve of the upland water meets the convex curve of the tidal water—rises higher and shifts seaward. The character of the two curves remains the same, but they become flatter and the surface NJ nearly level.

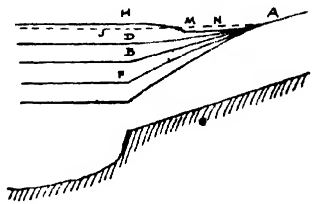


FIG 186.

Thus the time of high tide at M is later than at H. It is later for each point passed in going up the river towards A. A is the point where there is no tide; AH is the "tidal reach." Far below A, say at N, there is a point above which there is no up-river current. In going up the river the duration of the flood tide decreases and that of the ebb tide increases. The flood tide at M attains its greatest velocity soon after its commencement, the ebb tide towards its close. In a flood the water in the tidal reach may be ebbing continuously.

The length of the tidal reach is greater the greater the

range of the tide and the flatter the slope of the river. In any particular river it is greatest at spring tide. The discharge of a river may vary daily; the greater the discharge the more the rise of the river tends to keep pace with that of the tide and the shorter the tidal reach. If the river slope is flat the surface AH may, at high tide at H, be practically level. If, as in some rivers in the east of England, much silt is deposited in MH, A may be lower than H; the flow and the ebb are both impeded. On a longitudinal section of the river, the high-water line is for convenience shown as ANH. It is never high water at all points simultaneously. To show the actual state of affairs at various stages of the tide, series of lines must be drawn as in Fig. 187, where the firm lines show the flood and the dotted lines the ebb tide.

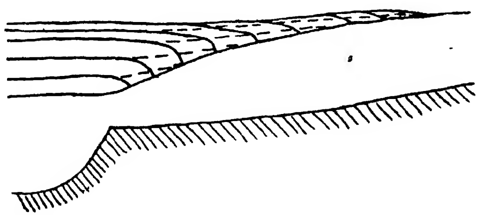


FIG 187

When, in a silt-bearing stream, there is alternately heading up and free flow, the tendency for deposit to occur is (p. 71) probably not much—if at all—greater than if there was no heading up. The upland water in the tidal reach not only has free flow, but draw-down. River floods may—as in non-tidal rivers—greatly reduce any tendency to silt.

If the sea water is free from silt its passage up and down the river is helpful. It may be charged with silt, or pick up silt in the river, but the whole volume of water which enters has to flow out again. On the whole, the tendency for its silt to be deposited in the river is due only to the period of "slack tide" about the time when the flow

ceases. Thus, as regards silting in the tidal reach, the sea water has little prejudicial effect if it is silt-laden, and a beneficial effect if it is not.

Sea water is heavier than fresh water by about 2.4 per cent., and this, to some extent, prevents their mixing. At all stages of the flood tide the tendency at the point where the fresh and salt water meet is for the fresh water to rise and the sea water to sink. When the tide begins to flow up the river there may be a low-level salt water landward current and a high-level fresh water seaward current. The two kinds of water mix eventually, and their temporary separation has no considerable effect on any tendency to scour or to silt.

A body of water included between any two cross-sections of the tidal reach may not reach the sea during the next ebb tide. In this case it will flow back with the next flood tide, and so be kept moving up and down, getting nearer, however, to the sea at each tide.

Works in Tidal Rivers. If any works are required in the tidal portion of a river, the principles to be followed in designing them are the same as if the river was non-tidal. Chap. V., Art. 3, applies to them. The river may be straightened or trained or dredged; frequently training and dredging are combined. Dredging in the portion of the river nearest the sea does not alter the water levels near the mouth but it alters them farther up; the tide comes up in greater volume, rises higher and extends farther up; the ebb is facilitated, and the low-water level lowered. Frequently the channel near the sea is narrowed by training walls in order to lower the bed. If the resulting deepening is not sufficient, the volume of tidal water farther up the channel is reduced, and this may be injurious. The proper course may be to continue the narrowing up-stream. If this is done, the width of channel in which deep water is to be maintained at high water, or which is to be kept free from deposit, is reduced in about the same proportion as the volume of tidal water.

Any weir, fixed or movable, which checks the flow of the

wide up a river checks it of course for a long distance back, perhaps to the mouth, and reduces the tidal flow. Old London Bridge used to obstruct the tide; its removal increased the range and was beneficial.

Tidal rivers generally widen out to some extent near their mouths, and thus become estuaries.

Art. 2. Tidal Estuaries. If, instead of a river of uniform width, there is a funnel-shaped estuary, the conditions above described are modified. An estuary is formed partly by waves which wear away the angles at the mouth of the river, but chiefly by the flow and ebb of the tides. The slope of the bed is usually much flatter than that of the river. The tidal reach extends farther up than in the case of a river, not only because of the greater difficulty experienced by the upland water in filling up the wide channel, but because of the funnel shape. The greater tendency of silt to deposit in an estuary as compared with a river channel is counteracted by the greater force of the tidal flow.

The ebb tide in an estuary does not always follow exactly the same course as the flood tide. The low-lying parts are filled first and emptied last, but the channels are not all continuous. A channel open at its lower end may be blocked at its upper end, and *vice versa*. Also at sharp bends the momentum of the water may cause differences in the paths traversed by the flowing and ebbing currents. Wherever there is a deep channel the water from the adjacent sandbanks tends, towards the close of the ebb, to flow cross-wise into the channel, and in doing this it to some extent washes down the banks into the channel.

Works in Tidal Estuaries. Estuaries, when shallow, offer great facilities for training. It used at one time to be said that any change which reduces the volume of tidal flow must be injurious. It would be injurious to restrict the mouth of the estuary, unless it were exceptionally wide, and leave the rest untouched. If the whole estuary is narrowed, and a suitable funnel shape preserved, the width to be kept open is, relatively to the size of the mouth, no greater than before, and the tide probably flows

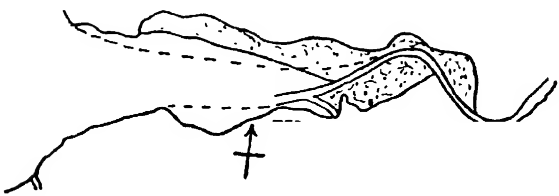
up as far as before, and rises to as high a level. The narrowing, if well designed, improves the shape of the estuary and causes increased scour. The effect of the upland water is also greater in the narrower channel. Improvements to estuaries are not, however, restricted to training. There are always one or more deep channels, and the best of these can be selected and improved by dredging. The channel should be one along which both the flood tide and the ebb tide will run. The above remarks as to training do not apply to a case in which there is a bar outside the mouth of the estuary. Training might check the scour at the bar. Bars are dealt with by dredging or by the construction of two jetties running out into the sea in continuation of the river banks. This matter is connected with littoral drift and is hardly within the scope of river engineering.

If an estuary is not funnel-shaped, if for instance it widens out very rapidly, the tidal flow is much less effective in keeping the channel open. In this case, training works, which would give the necessary funnel shape, are indicated rather than dredging. If an estuary is narrow at the entrance, the flow is much less powerful, unless the narrow part is of greater depth, but even then the force of the tide is reduced owing to the change in the shape of the channel.

The bed of an estuary may be of such soft or sandy material that a dredged channel would be likely to be quickly filled up again by the slipping in of material at the sides. In such a case an untrained channel can only be kept open to its full depth by constant dredging, and probably the best course is to construct a trained channel, although it may be more expensive than in the case of a harder channel, because of the depth to which the foundations of the walls must be sunk into the soft bed. Also, if the bed of the estuary is constantly shifting, a dredged channel alone will not succeed, and training must be resorted to. Again, the bed may be of such hard material that training walls would not cause it to scour. In this case a channel should be dredged and need not be trained.

For the great body of intermediate cases in which the deep channel can be formed either by dredging or training, both methods can be adopted. A common plan is to train the upper part and to dredge the lower part where the estuary is wider and the training walls would be more exposed to the waves.

When an estuary is thus partly trained, the deepening due to the training does not extend far beyond the point where the walls terminate. The deposit of material along the sides of the estuary may, however, extend some distance farther down in places where the tide can no longer have free play. This occurred in the Seine estuary (Fig. 188). The authorities of Havre, which lies at one side of



the estuary not far from its mouth, feared that if the training walls shown by the firm lines were brought farther down, the deposits might extend to their neighbourhood. The reduction in the capacity of the estuary, due to the deposits, caused it to become filled up more quickly, and the time of high water at Havre was advanced. The dotted lines show a good arrangement of training walls proposed by Harcourt.

It is always feasible to carry training walls right through an estuary, or at least down to a point where deep water is reached, and if a proper funnel shape is given to the channel the reduction of the tidal flow and silting up of the spaces behind the walls need not cause any trouble. Training the complete estuary was carried out in the case of the Tees, where, however, the estuary was not of great length, and was not of a good shape for keeping itself open. Any

affluents entering the estuary can be provided with separate trained channels. Difficulty may, however, arise if there are towns which would be shut off from the estuary by the silt banks.

Generally the line selected for the trained or dredged channel should, though it must be as short and direct as possible, coincide as nearly as possible with that which the water naturally tends to keep open. This may be toward one side of the estuary or the other, according to the direction from which the tidal wave approaches. In the case of the Dee, the best line was not adopted, attention having been chiefly given to the question of silting up the spaces outside the walls and so reclaiming land, a matter which should always be treated as of quite secondary importance. Training walls in estuaries are generally built only up to half-tide level. Were it not for the expense they might be built up to high-water level. In the Seine estuary the walls were made of blocks of chalk.

Whether a trained channel will keep itself open or will need periodical dredging depends on the amount of silt in the water and on its velocity and depth. The question must be worked out and calculated as in the case of a non-tidal river.

The estuary of the Mersey differs from most others. Towards the mouth, near Liverpool, it is narrow and it widens out farther inland. The tides, running through the narrow portion, to fill up the large inland basin and to empty it again, keep the narrow part scoured to a great depth. It was proposed to train the wide portion for the Manchester Ship Canal. The training would, no doubt, have succeeded, but, owing to the silting up of the greater part of the estuary, the scouring near Liverpool would have been very greatly reduced and serious damage done to that port.

Art. 3. Training of the Rangoon River.[1] This work was carried out in 1912–14. Rangoon (Fig. 189) is 28 miles

[1] *Proc. Inst. C.E.* Vol. 202, p. 143 (Buchanan).

from the mouth of the river. The anchorage extends along the left bank below Mower's Point. In 1824 the right bank of the river was approximately where the wall *ac* now is. Since then the river had been eroding its right bank while sandbanks formed on the left. If the process had continued and extended downstream, the town would have been cut off from the river. Moreover, the stream crossed over from Mower's Point to the left bank, where it

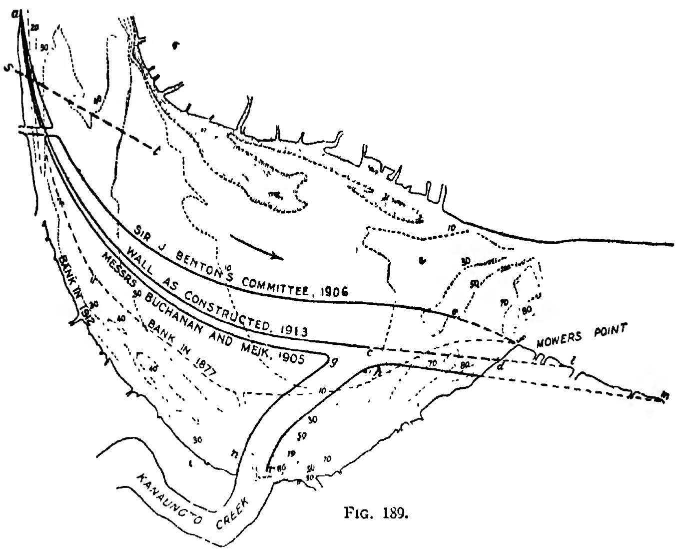


FIG. 189.

undermined jetties and piers—revetment work being necessitated—and the cross currents in the anchorage tended to limit the number of fixed moorings.

The tidal reach extends up to 30 miles above Rangoon. In the dry season the average fresh water discharge, Q, is 19,000 c. ft. per second, and the average total discharge—fresh and tidal waters—at neap and spring tides 530,000 and 887,000 c. ft. per second respectively. In the monsoon season Q is 139,000 c. ft. per second and the maximum total discharge—spring tide and highest flood—about

1,000,000 c. ft. per second. The average surface velocities during flood and ebb were 6 and 6.65 ft. per second respectively in spring tides and 3.55 and 3.7 ft. per second in neap tides.

The wall constructed cost a million sterling. It has an extension *cd* 2,700 ft. long at a lower level. At Mower's Point the top of the wall is 30 ft. below low water so that the river Kanaungto Creek can flow over it. The proposal of 1905 included heavy expenditure in cutting off Mower's Point. Sir John Benton, Inspector-General of Irrigation in India, was chairman of a committee which in 1906 objected to this expenditure and to other items. More recently it was suggested that protection of the right bank of the river, as it was in 1912, would have sufficed; but it would not have put a stop to the cross currents. It was also suggested that a spur *st* would have sufficed. The difficulty of constructing it would have been great and its effect doubtful.

The training wall is of rubble and it was built on a mattress which cost £111,436. If the mattress had been omitted the above sum would have covered the cost of stone a yard deep extending over the whole area of the foundation of the wall. Nothing but stone was used in the training works for the great Indian railway bridges and for the Indian river weirs. Sir George Buchanan proposed the mattress and adhered to his proposal. Sir John Benton as steadily opposed it, but in order that a deadlock might be avoided the design with a mattress was sanctioned. The top of the stone wall was at level 95 ft. The rush of water over the wall prevented the silting up of the space behind the wall. It did not shift the stones. To stop this rush there was added for a length of 9,200 ft., 9 ft. of "permeable" wall with vertical faces of reinforced concrete slabs (3 ft. × 3 ft. × 4 in.) placed on edge and held together by 1-in. steel tie rods, rubble being filled in between them. This upper wall cost only £13,665. A proposal to use permeable bamboo screens supported by steel piles—driven when the work on the wall was commenced—was rejected.

It would have cost £140,000 and the piles would have seriously obstructed the barges bringing stone. In the design of 1905 the wall consisted of a series of mattresses and stone in alternate layers. It would have been impossible to get the wall to the correct profile. Moreover, the mattresses would have been destroyed by the teredo.

In the actual wall each mattress was 125 ft. long, 75 or 80 ft. broad and 3 ft. thick and was built on a pitched slipway which was wholly submerged in spring tides. The mattresses were of brushwood. Ropes of it, 6 inches in diameter—tightly bound by galvanized wire at intervals of 1½ ft.—were laid 3 ft apart. A similar row was laid over them and at right angles, and the crossings tied alternately with wire and tarred rope, the latter being left long. Over this "grill" were three 6-inch layers of brushwood—each at right angles to the next—and over this a second grill which was secured by the tarred ropes. Stakes were also driven through the mattress, passing through the grill ropes. The mattresses were sunk by being loaded with stone. They were laid for a width of 230 ft. over the whole length of the wall and, as soon as laid, more stone was dropped on them, from hopper barges, along the centre of the wall.

Simultaneously with the above work a suction dredger made a series of cuts along the line of the new channel outside the wall, the material being conveyed by a pipe line to the back of the wall.

Where the depth of water was small the width of mattress was greater than that of the wall. The excess formed an apron which to some extent sank and revetted the bank as scour occurred. Eventually the apron became covered with silt. It was reported that 6 inches of silt would prevent damage by the teredo.

The work succeeded. A deep navigable channel was formed in front of the wall and the sandbanks near the left bank were reduced. The whole space behind the wall was silted up to high water level, and in 1920 was covered with dense scrub and bushes.

At the wharves on the left bank opposite and just above Mower's Point dredging had been necessary. Less was required after the completion of the work.

Note. The figures on the plan are soundings at *l.w.o.s.t.*, which was 90. *H.w.o.s.t.* was 109. The permeable wall was in three tiers and its top at 104.

APPENDIX A.

NOTES TO CHAPTER II (RAINFALL, ETC.).

Heavy Falls (p. 14). In Rome, on 19th October, 1o:z, 8 inches of rain fell in 24 hours. At Columbus, Ohio, a fall in 5 minutes was at the rate of 5.6 inches per hour, and at Adelaide, Australia, in 6 minutes at the rate of 5.65 inches per hour.[1]

In India in the Deccan a fall in 5 minutes may be at the rate of 9 inches per hour.[2]

Rain Gauges (p. 23). In India one rain gauge for 50 or 100 square miles is often as much as can be aimed at.

Yield of Catchments (p. 30). In estimating the yield of a catchment area a complete geological survey may assist greatly. The following figures for forest reserves in Malaya are supplementary to the table on p. 34, the mean annual rainfall being 92.5 inches :—Ayer Kuning 300 acres, yield per cent. 23.4; Sungei Bharu 202 acres, yield per cent. 22.8.[3]

NOTES TO CHAPTER III (ACTION OF STREAMS).

Silting and Scour (p. 52). Observations on two Bavarian rivers have shown that for a given gauge reading the silt charge is greater when the river is falling than when it is rising.[4]

Floods and Silt. In a recent treatise it is stated that silt is deposited chiefly by the rising flood, and that the flow is then obstructed by the water downstream of it. Presumably a misprint has occurred. The obstruction, of course, occurs in a falling flood. The upstream supply and water level fall off before those farther down are affected; the surface slope flattens and the flow becomes slack because of the deeper water downstream.

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[1] *Engineering Abstracts* (Inst of Civil Engineers). No 32, p 18[illegible].
[2] *Proc. Inst C.E.* Vol 224, p 136 (Stonebridge).
[3] *Eng. Assoc. Malaya* Vol. 4, pp 43, 48 and 70 (Morgan and Boissier).
[4] *Le Bautechnek.* Vol 7. pp 525 and 600.

Shifting Rivers (p. 67). The Sacramento River in California is a good example of such a river in countries other than India. It is obvious that in any such river there may in any reach be "retrogression of levels." Inferring the flood level at any point is not very simple (p. 6), but before designing a weir or other structure some definite figure must be arrived at.

Notes to Chapter IV (Control of Scour).

Bank Protection. On the Sacramento River spurs of tree trunks—stripped of branches and lashed together—were run out at right angles to the bank and secured by cast-iron screws sunk in the bank and in the river bed.[1]

On the Missouri (p. 98) emergency work during floods consisted in placing old steel coal-wagons in the water about 20 ft. from the bank and brush between the wagons and the shore, weighted with sand-bags and riprap. In many places the bank is protected between the retards by cribbing filled with stone.[2] Drifting material is apt to collect against the dykes and thus they may be carried away.

Bank protection on the Missouri has also been effected by mattresses and mud cells (p. 94). A heavy "loose fascine" mattress was constructed for 1,400 ft. along the foot of the bank. It was made of rolls of willow brush, 14 in. thick; these were wired to poles about 20 ft. long, placed parallel to the bank and 8 ft. apart. Anchor cables —½-in. galvanized strand—ran up the bank through trenches to "deadmen," 8 ft. back from the top of the bank and 4 ft. below the surface. During construction the mattress floated and was then sunk by stone. Mud-cells 6 ft. square, of poles tied by ⅜-in. cable, were made on the mattress, and at each 3 ft. of height they were stepped back to form a slope of 1 in 2. A fender mattress, or else "brush decking," covered the cells. Wing walls, 48 ft. apart, of mud-cells 16 ft. wide, connected the structure with the bank. After two periods of high water it was completely

[1] *Engineering News Record* Vol. 97, p 190

[2] *Engineering News Record.* Vol. 93, p 372

filled with silt. Willows grew densely on the top. The work resists the action of floating ice as well as that of scour.[1]

Flexible mattresses (p. 97) made of reinforced concrete have for long been used in Japan on deep rivers.[2]

Spiders (p. 99) have lately been made of poles. Several spiders are attached to one cable—which anchors them to the bank—and rolled into the stream, forming a spur. If the spurs are close together the whole forms an entanglement and the ends of the poles are caught in the bed.[3]

Cribs of reinforced concrete filled with stones and hinged together and anchored are used in some Italian torrents to form short spurs or for a continuous lining of the bank.

On the Po, "bolsters" about 13 ft. long are made. A sheath of willows, is filled with stone the ends of the willows tied together and the whole bound with galvanized wire. Five bolsters fastened together are lowered from the side of a barge.

For spurs or training walls a line of piles 2 m. apart and 2 m. high—above the bed—is driven so as to enclose a rectangle, and bolsters are laid round the outside of the line forming a slope of 2 to 1. Sand is filled inside the line and into it an inner ring of piles, 4 m. from the first, is driven and with their tops 2 m. higher. A second lot of bolsters and sand filling is added. Further lines can be added if necessary to gain the required height.[4]

It has been suggested that in order to secure silt deposit by a spur, only the upper water should be obstructed by the spur, so that the carrying and rolling of the heavier silt may go on. But the rush under the spur probably prevents deposit (p. 341).

Notes to Chapter V (Works for Control of Streams).

Diversions of Streams (p. 102). The need for controlling scour in diversions has been mentioned on pp. 103 and 104.

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[1] *Engineering News Record*. Vol 92, p 1,024

[2] *Engineering News Record*. Vol. 67, p 922.

[3] *Public Roads*. Vol 7, p 53

[4] *Annali dei Lavori Pubblici*. Vol. 66, p. 414, and Vol 67, p 841.

It may be convenient to build a weir "in the dry" at some point, say C, on a diversion AB (Fig. 40, p. 103). If, as is likely, it is desired to preserve the original slope S of the stream, the slopes in BC and CA will be S, and there is a fall (Fig. 119, p. 249) at C. If the channel is a large one and the diversion is merely a leading cut, its enlargement may take time. While it is going on there is heading up at B and—since the weir is of full size—draw-down in BC; also heading up at C below the weir and draw-down in CA. (*Hydraulics*, Figs. 126, 127 and 129). The scour will tend to be heaviest at A, and it may extend somewhat down-stream of A because of the momentum of the stream. If CA deepens itself—whether it widens itself or not—steps may be necessary to prevent the undermining of the down-stream bed protection of the weir, and even of its floor.

NOTES TO CHAPTER VI (ARTIFICIAL CHANNELS).

Suitable Velocities for Earthen Channels. It has been remarked (p. 178) that local knowledge and judgment are necessary. There are no absolute figures. In America an enquiry was made from several engineers of the Reclamation Service as to the velocity which they considered suitable—in the cases of straight canal reaches—for each kind of soil. The figures given ranged in the cases of alluvial silt from 2 to 4 ft. per second, fine loam 1.8 to 2.75 ft. per second, fine sand 1.5 to 2.5 ft. per second, coarse sand 2 to 3 ft. per second, fine gravel 2.8 to 4 ft. per second, coarse gravel 3.5 to 6 ft. per second.[1]

Losses of Water from Channels (p. 143). When, as in the Indian Deccan, the soil is not merely alluvial but contains stratified rocks, some of which such as "murum" (p. 188) are highly porous, the leakage of water may be great, especially when a porous stratum is inclined. It may be desirable to insert in the banks puddle walls (p. 307) carried down to an impermeable stratum. If such stratum is inclined, the wall can be only in the bank towards which

[1] *Proc. Am Soc. C.E* Vol 51, p 1,397.

the stratum slopes.[1] A local increase of the cross-section of a channel by widening or deepening or both, causes reduced velocity and may tend to reduce loss of water. It will probably do so if silt deposits and forms a lining. It has been suggested that experiments be made on canal watercourses in India. The expense would be moderate.

In the Gang Canal, which forms part of the Sutlej Valley Irrigation Project, a canal 84 miles long with bed width 52 ft. and depth of water 9 ft., has been lined with concrete. This may perhaps save 300 or 400 c. ft of water per second.[2]

NOTES TO CHAPTER VIII (WEIRS, BRIDGES, ETC.).

Shapes of Piers (p. 276). Recent investigations (see *Hydraulics*, Notes to Chap. IV) show that the pier described when it has pointed ends gives slightly more heading up than a fish-shaped pier. The matter is of importance only when heading up has to be reduced to a minimum.

Weirs on Porous Soil (p. 242). Syphons as well as weirs are subject to the dangers mentioned. In the cases of the large syphons carrying torrents under the Upper Jhelum Canal (p. 278) the floor—built on the torrent bed—of the syphon is usually dry. The canal water level is considerably higher and in several cases piping occurred under the wing walls and the floor.[3] The issuing water carried sand and settlements occurred in pitching, floors and wing walls. The remedies adopted were lines of sheet piling across the bed of the torrent near the upstream and downstream edges of the floor, and the provision of drains with strainers whereby some water could escape without carrying solids with it.

It was found that along the line of gradient the height to which water rose in a pressure pipe increased as the pipe was driven deeper. It may be that the lower lines in Figs. 116 and 117 (p. 242) are not exactly as shown and have flatter gradients. They are probably in wetter soil than the upper lines.

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[1] *Proc Bombay Engineering Congress.* Vol 17, Paper 112 (Inglis).
[2] *Proc Punjab Engineering Congress, 1926* (Jefferis)
[3] *Punjab Engineering Congress, 1930.* (Ajudhia Nath Khosla). Papers received shortly before going to press

Whatever the structure, the case is not always very simple. The hydraulic gradient is not necessarily a straight line, nor is it likely to be the same in every part of a structure. Piping may have undermined a floor for some distance, but may have choked the outlet farther down. The floor is then partly supported by the water pressure under it and any reduction of the pressure may cause damage. In such a case the gradient is flattened in its upper part; this was the case in the syphons. In one of the syphons the temporary closure of the canal was followed by cracks in the floor. On one occasion the canal over the syphon was puddled, on bed and both inside slopes, for a length of 150 ft., but no benefit resulted. The water table all along the canal had permanently risen (p. 21) and the water in the canal—except for the 150 ft.—could still supplement it.

The great superiority of a line of wells to a line of sheet piling is obvious. The former can support the structure over it. The gaps between the wells can be cleared out deeply by means of the water jet and concrete put in.

Damage to—or destruction of—a weir where water is being headed up may occur in the low-water season by uplift or by undermining—both due to piping; but the original cause is quite likely to have been heavy downstream scour in floods. Cross flow has been mentioned (p. 251) and damage to two great weirs (p. 253).

Standing waves (p. 240) are fully discussed in *Hydraulics*. In a deep stream when the wave is high the pressure on the floor—or glacis—under it is much greater than just upstream of it. In the Merala weir in an exceptional flood, the wave was 8 ft. high. Water coming through cracks caused an uplift of a foot in part of the upper layer of the glacis, the lower layers being undisturbed. This, as well as the severe disturbance caused by the wave, is why the wave position must be well up the glacis where the water level is high and the structure heavy. The upper layer can be in compartments (p. 251) and well connected to the lower work.

In the case of damage to the Islam weir on the Sutlej in 1929, a government committee reported that the cause was retrogression of levels. The shutters of the Rasul weir (p. 253) were removed in 1922 or thereabouts and the weir crest raised several feet. The reasons for this are not known. The weir was severely damaged in 1929. The flood was altogether abnormal and was 4 ft. higher than what had been the accepted flood level. If the weir had not been altered the shutters would have been down and the passage of the flood greatly facilitated.

Both in hydraulics and in engineering, weirs require very great attention. Care is necessary not only in design and construction but in working and maintenance (p. 285) and particularly in heading up the water—after a flood—without close examination.

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